PROFESSIONAL PROJECT

ARCH 6500

SPRING 1986

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INTRODUCTION

This professional project is for the fulfillment of the requirement of Architecture 6500 (Special Problems). With the completion of this project, the educational coursework required for the degree of Masters of Architectural Engineering at Oklahoma State University will be satisfied.

In the fall semester if 1983 my advisor, Professor Louis O. Bass, and I decided that the structural design of a four-story reinforced concrete building would fulfill the requirements of Architecture 6500 and would provide me with practical experience in the design of a reinforced concrete building. Professor Bass suggested that I use the same project which I architecturally designed and detailed in Architecture 5998 for my professional project. Bryan High School is the name of the project which I designed and detailed in Architecture 5998 in the fall semester of 1982. My design of Bryan High School was one of the very few one-story projects in my class and was therefore not adaptable to multi-story design. I alternately selected the project of one of my classmates from Architecture 5998 as the one I would use for my professional project. After showing the project to both Professor Bass and Professor Orr it was agreed that this particular project would be adequate for me to use for my professional project. It was further decided that column locations would remain the same and the 2'-0" clearance requirement of the architectural drawings would be adhered to.

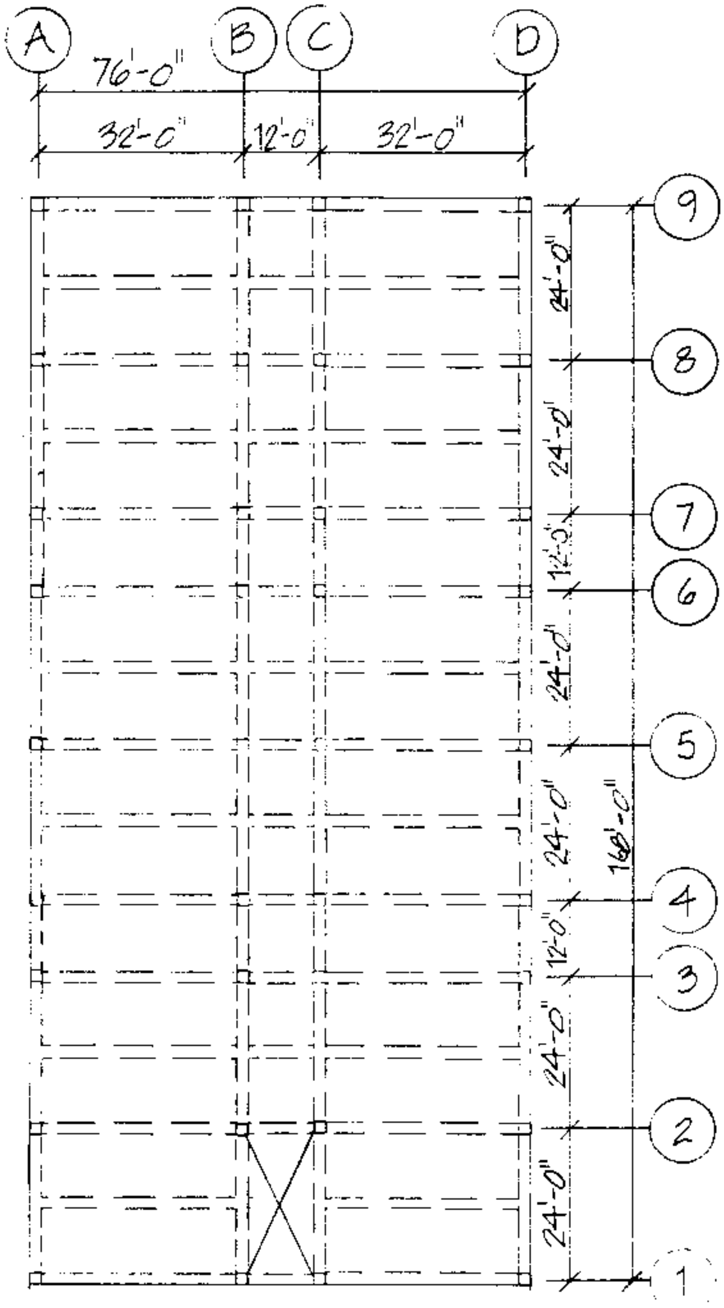
For the structural design of Bryan High School several structural systems were considered. The structural systems considered were:

1) Beam and Slab
2) Pan Joist
3) Waffle Slab
4) Flat Plate
5) Flat Slab

The waffle slab, flat plate, and flat slab systems were eliminated from consideration due to the fact that the building has non-uniform column spacings. The beam and slab system was chosen over the one-way joist system because it forms nine continuous frames in the north-south direction and four continuous frames in the east-west direction. These frames were utilized in resisting the lateral loads applied to the building. With the addition of intermediate beams at the midpoints of the (east-west spanning) girders, fourteen even slab spans were formed. Framing the building in this way provided repetition in both the design and construction of the building.

The foundation system consists of piers and grade beams. An allowable soil bearing pressure of 10 kips per square foot at a depth of 20 feet below grade was assumed.

STRUCTURAL PLAN – TYPICAL FLOOR



SCALE: 1" = 30'-0"

SLAB THICKNESS

TABLE 9.5 (a) OF ACI 318-83 GIVES RECOMMENDED SLAB SPANS AND THICKNESSES AS FOLLOWS:

TABLE 9.5 (a)
MINIMUM THICKNESS

MEMBER	ONE-END CONTINUOUS	BOTH-ENDS CONTINUOUS
SOLID ONE-WAY SLAB	$\frac{L}{24}$	$\frac{L}{28}$

* W_c = 145 PCF
F_Y = 60000 PSI

PLACING THE INTERMEDIATE BEAMS 12'-0" O.C. (MIDPT. OF 24'-0" SPANS), THE SLAB THICKNESS EQUALS:

$$\frac{h}{28} = \frac{(12\text{ FT})(12\ \frac{\text{IN}}{\text{FT}})}{28} = 5.14''$$

THEREFORE USE A <u>5¼"</u> THICK SLAB

SLAB LOADS

L.L. FLOORS 80 PSF
ROOF 20 PSF

$$\text{SLAB WEIGHT} = \left(150\ \frac{\text{LB}}{\text{FT}^3}\right)(5.25\text{ IN})\left(\frac{1\text{ FT}}{12\text{ IN}}\right) = 65.625\ \frac{\text{LB}}{\text{FT}^2}$$

$$W_D = \left(65.625\ \frac{\text{LB}}{\text{FT}^2}\right)(1.4) = 92\ \frac{\text{LB}}{\text{FT}^2}$$

$$\left.\begin{aligned} W_L &= \left(80\ \tfrac{\text{LB}}{\text{FT}^2}\right)(1.7) = 136\ \tfrac{\text{LB}}{\text{FT}^2} \\ W_T &= 92 + 136 = 228\ \tfrac{\text{LB}}{\text{FT}^2} \end{aligned}\right\}\ \text{FLOORS}$$

$$\left.\begin{aligned} W_L &= \left(30\ \tfrac{\text{LB}}{\text{FT}^2}\right)(1.7) = 51\ \tfrac{\text{LB}}{\text{FT}^2} \\ W_T &= 92 + 51 = 143\ \tfrac{\text{LB}}{\text{FT}^2} \end{aligned}\right\}\ \text{ROOF}$$

☐ MEMBER SHEARS & MOMENTS

IN LIEU OF A MORE ACCURATE METHOD OF FRAME ANALYSIS, THE FOLLOWING APPROXIMATE MOMENTS AND SHEARS WILL BE USED FOR PRELIMINARY DESIGN OF CONTINUOUS BEAMS AND ONE-WAY SLABS (ACI 318-83 8.3.3).

	1–2	2–3	3–4	4–5	5–6	6–7	7–8	8–9
SHEAR @ L. SUPPORT	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{1.15WL}{2}$
SHEAR @ R. SUPPORT	$\frac{1.15WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$
−MOM. @ L. SUPPORT	$\frac{-WL^2}{24}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{10}$
−MOM @ R. SUPPORT	$\frac{-WL^2}{10}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{24}$
+MOM @ MIDSPAN	$\frac{+WL^2}{14}$	$\frac{+WL^2}{16}$	$\frac{+WL^2}{16}$	$\frac{+WL^2}{16}$	$\frac{+WL^2}{16}$	$\frac{+WL^2}{16}$	$\frac{+WL^2}{16}$	$\frac{+WL^2}{14}$

	A–B	B–C	C–D
SHEAR @ L. SUPPORT	$\frac{WL}{2}$	$\frac{WL}{2}$	$\frac{1.15WL}{2}$
SHEAR @ R. SUPPORT	$\frac{1.15WL}{2}$	$\frac{WL}{2}$	$\frac{WL}{2}$
−MOM. @ L. SUPPORT	$\frac{-WL^2}{24}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{10}$
−MOM. @ R. SUPPORT	$\frac{-WL^2}{10}$	$\frac{-WL^2}{11}$	$\frac{-WL^2}{24}$
+ MOM. @ MIDSPAN	$\frac{+WL^2}{14}$	$\frac{+WL^2}{16}$	$\frac{+WL^2}{14}$

INTERMEDIATE BEAMS (FLOORS)

- TABLE 9.5(a) OF ACI 318-83 IN ORDER TO CONTROL DEFLECTION RECOMMENDS BEAM SPANS AS FOLLOWS:

TABLE 9.5 (a)
MINIMUM DEPTH

MEMBER	ONE - END CONTINUOUS	BOTH - ENDS CONTINUOUS
BEAMS OR RIBBED ONE-WAY SLABS	$\frac{L}{18.5}$	$\frac{L}{21}$

* W_C = 145 PCF
F_Y = 60000 PSI

- INTERMEDIATE BEAMS 32'-0" SPAN

$$\frac{L}{18.5} = \frac{(32\ FT)(12\ IN/FT)}{18.5} = 20.8'' \quad \therefore \text{TRY } H = 24\ IN. \quad (d = 21\ IN.)$$

$$W_{SLAB} = (5.25/12\ FT)(0.150\ KCF)(12\ FT) = 0.788\ KLF$$

$$W_{BEAM} = \text{ASSUME SELF WT.} = 0.500\ KLF = 0.500$$

$$W_{DL} = 1.288\ KLF$$

$$W_{L.L.} = (0.080\ KSF)(12\ FT.) = 0.960\ KLF$$

$$W_{ULT.} = (1.4)(1.288\ KLF) + (1.7)(0.960\ KLF) = 3.435\ KLF$$

$$\text{MAX. NEGATIVE MOMENT} = \frac{W_u L^2}{10} = \frac{(3.435\ KLF)(32\ FT.)^2}{10} = 351.744\ K\text{-}FT.$$

$$\text{MAX. POSITIVE MOMENT} = \frac{W_u L^2}{14} = \frac{(3.435\ KLF)(32\ FT.)^2}{14} = 251.246\ K\text{-}FT.$$

$$\text{TO CONTROL DEFLECTION} \quad \rho_{DESIRED} = 0.5(.75\rho_{BAL}) = (0.5)(0.0214) = 0.0107$$

$$m = \frac{f_Y}{0.85 f'_c} = \frac{60\ KSI}{0.85(4\ KSI)} = 17.647$$

$$R_u = \rho f_Y (1 - \tfrac{1}{2}\rho m) = (0.0107)(60000\ PSI)[1 - (.5)(0.0107)(17.647)] = 581\ PSI$$

$$-M_n = \frac{-M_u}{\phi} = \frac{351.744\ K\text{-}FT}{0.9} = 390.827\ K\text{-}FT.$$

$$\text{REQ'D } bd^2 = \frac{M_n}{R_u} = \frac{(390.827\ K\text{-}FT)(12000\ LB\text{-}IN/K\text{-}FT)}{581\ PSI} = 8072.151\ IN^3$$

$$d \approx 21\ IN. \quad \therefore \quad b = 18.304\ IN. \qquad \text{TRY } b = 18\ IN.$$

$$R_{u\,(REVISED)} = \frac{M_n}{bd^2} = \frac{(390.827\ \text{K-FT})(12000\ \text{LB-IN / K-FT})}{(18\ \text{IN})(21\ \text{IN})^2} = 591\ \text{PSI}$$

$$\rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2mR_u}{f_y}}\right) = \frac{1}{17.647}\left(1-\sqrt{1-\frac{(2)(17.647)(591)}{60000}}\right) = 0.0109$$

$$A_s = \rho bd = (0.0109)(18\ \text{IN.})(21\ \text{IN.}) = 4.119\ \text{IN}^2$$

CHECK SHEAR:

$$V_u = \phi V_n$$
$$V_n = V_c + V_s$$

$$V_{u\,(MAX)} = \frac{1.15\,wL}{2} = \frac{(1.15)(3.435\ \text{KLF})(32\ \text{FT})}{2} = 63.204^K$$

$$V_c = 2\sqrt{f'_c}\,bd = (2)(\sqrt{4000\ \text{PSI}})(18\ \text{IN})(21\ \text{IN.}) = 47.814^K$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{63.204^K}{0.85} - 47.814^K = 26.544^K$$

$$4\sqrt{f'_c}\,b_w d = (4)(\sqrt{4000\ \text{PSI}})(18\ \text{IN})(21\ \text{IN.}) = 95.627^K > V_s \quad \therefore \quad S_{MAX} = \frac{d}{2} = 10.500\ \text{IN.}$$

$$V_s = \frac{A_v f_y d}{s} \quad \therefore \quad s = \frac{A_v f_y d}{V_s} = \frac{(A_v)(40\ \text{KSI})(21\ \text{IN})}{26.544^K} = 31.646\,A_v$$

STIRRUP SIZE $\longrightarrow$ #3 @ 6.962 IN. O.C.
#4 @ 12.658 IN. O.C. $\longleftarrow$ N.G. $> d/2$

USE 18"x 24" INTERMEDIATE BEAMS (FLOORS)

EXTERIOR BEAMS (FLOORS)

EXTERIOR BEAMS 24'-0" SPAN

ASSUME DEPTH = H = 24 IN. (d = 21.5 IN.)

LOADS: 1) CONCENTRATED LOAD AT MIDSPAN FROM INTERMEDIATE BEAMS
2) UNIFORM LOAD FROM FACADE
3) UNIFORM LOAD FROM SELF WEIGHT

LOAD 1) TO USE THE PRELIMINARY DESIGN COEFFICIENTS OF ACI 318-83 8.3.3, AN EQUIVALENT LOAD PER FOOT THAT WILL GIVE THE SAME MAXIMUM MOMENT AS THE CONCENTRATED LOAD AT MIDSPAN MUST BE COMPUTED. THIS IS DONE BY EQUATING THE MAXIMUM MOMENT DUE TO THE CONCENTRATED LOAD TO THE MAXIMUM MOMENT DUE TO A UNIFORM LOADING AND SOLVING FOR THE FACTORED LOAD PER FOOT w_u:

$$\frac{PL}{8} = \frac{w L_n^2}{10} \quad \therefore\ w = \frac{10PL}{8L_n^2}$$

CONCENTRATED LOAD $P = (16\text{ FT})(3.435\text{ KLF}) = 54.960^{K}$

$$\text{EQUIVALENT LOAD/FT} = \frac{(10)(54.960^{K})(24\text{ FT})}{(8)(24\text{ FT})^2} = 2.862\text{ KLF}$$

LOAD 2) UNIFORM LOAD FROM FACADE = (0.595 KLF)(1.4) = 0.833 KLF

LOAD 3) UNIFORM LOAD SELF WT (ASSUMED) = (0.300 KLF)(1.4) = 0.420 KLF

w_u = 4.115 KLF

$$\text{MAXIMUM NEGATIVE MOMENT} = \frac{w_u L^2}{10} = \frac{(4.115\text{ KLF})(24\text{ FT})^2}{10} = 237.030\text{ K-FT.}$$

$$\text{MAXIMUM POSITIVE MOMENT} = \frac{w_u L^2}{14} = \frac{(4.115\text{ KLF})(24\text{ FT})^2}{14} = 169.303\text{ K-FT.}$$

$\rho_{DESIRED} = 0.0107$
$m = 17.647$
$R_u = 581$ PSI

$$M_n = \frac{M_u}{\phi} = \frac{(237.030\text{ K-FT.})}{(0.9)} = 263.367\text{ K-FT.}$$

$$\text{REQ'D } bd^2 = \frac{M_n}{R_u} = \frac{(263.367\text{ K-FT})(12000\text{ LB-IN/K-FT})}{581\text{ PSI}} = 5439.594\text{ IN}^3$$

$d \approx 21.5$ IN. $\therefore$ b = 11.768 IN. TRY b = 12 IN.

$A_s = \rho bd = (0.0107)(12\text{ IN.})(21.5\text{ IN.}) = 2.761\text{ IN}^2$

$$R_{u(\text{REVISED})} = \frac{M_n}{bd^2} = \frac{(263.367\text{ K-FT})(12000\text{ LB-IN/K-FT})}{(12\text{ IN})(21.5\text{ IN.})^2} = 597\text{ PSI}$$

$$\rho_{\text{REVISED}} = \frac{1}{m}\left(1-\sqrt{1-\frac{2mR_u}{f_y}}\right) = \frac{1}{17.647}\left(1-\sqrt{1-\frac{(2)(17.647)(597)}{60000}}\right) = 0.0110$$

$A_s = \rho bd = (0.0110)(12\text{ IN})(21.5\text{ IN.}) = 2.845\text{ IN}^2$

CHECK SHEAR:

$V_u = \phi V_n$
$V_n = V_c + V_s$

$$V_{u(\text{MAX})} = \frac{1.15wL}{2} = \frac{(1.15)(4.115\text{ KLF})(24\text{ FT})}{2} = 56.787^K$$

$$V_c = 2\sqrt{f'_c}\,bd = (2)(\sqrt{4000\text{ PSI}})(12\text{ IN.})(21.5\text{ IN.}) = 32.635^K$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{56.787^K}{0.85} - 32.635^K = 34.173^K$$

$$4\sqrt{f'_c}\,bwd = (4)(\sqrt{4000\text{ PSI}})(12\text{ IN})(21.5\text{ IN.}) = 65.269^K > V_s \quad \therefore\ S_{max} = \frac{d}{2} = 10.750\text{ IN.}$$

$$V_s = \frac{A_v f_y d}{s} \quad \therefore\ s = \frac{A_v f_y d}{V_s} = \frac{(A_v)(40\text{ KSI})(21.5\text{ IN.})}{34.173^K} = 25.166\,A_v$$

STIRRUP SIZE $\longrightarrow$ #3 @ 5.537 IN. O.C.
#4 @ 10.066 IN. O.C.
#5 @ 15.603 IN. O.C. $\leftarrow$ N.G. > d/2

USE 12" x 24" EXTERIOR BEAMS (FLOORS)

☐ INTERIOR BEAMS (FLOORS)

☐ INTERIOR BEAMS 24'-0" SPAN

ASSUME DEPTH = H = 24 IN. ($d = 21.5$ IN.)

LOADS: 1) CONCENTRATED LOAD AT MIDSPAN FROM INTERMEDIATE BEAMS
2) UNIFORM LOAD FROM SELF WEIGHT

LOAD 1) $\frac{PL}{8} = \frac{wL^2}{10} \therefore w = \frac{10PL}{8L_n^2}$

CONCENTRATED LOAD $P = (22\text{ FT})(3.435\text{ KLF}) = 75.570^k$

EQUIVALENT LOAD/FT $= \frac{(10)(75.570^k)(24\text{ FT})}{(8)(24\text{ FT})^2} = 3.936$ KLF

LOAD 2) UNIFORM LOAD - SELF WT. (ASSUMED) $= (0.300\text{ KLF})(1.4) = \underline{0.420\text{ KLF}}$

$W_u = 4.356$ KLF

MAXIMUM NEGATIVE MOMENT $= \frac{W_u L^2}{10} = \frac{(4.356\text{ KLF})(24\text{ FT})^2}{10} = 250.906$ K-FT.

MAXIMUM POSITIVE MOMENT $= \frac{W_u L^2}{14} = \frac{(4.356\text{ KLF})(24\text{ FT.})^2}{14} = 179.218$ K-FT.

$\rho_{DESIRED} = 0.0107$

$m = 17.647$

$R_u = 581$ PSI

$M_n = \frac{M_u}{\Phi} = \frac{(250.906\text{ K-FT})}{(0.9)} = 278.784$ K-FT.

REQ'D $bd^2 = \frac{M_n}{R_u} = \frac{(278.784\text{ K-FT})(12000\text{ LB-IN/K-FT})}{581\text{ PSI}} = 5758.026\text{ IN}^3$

$d = 21.5$ IN. $\therefore$ $b = 12.457$ IN. TRY $b = 12$ IN.

$A_s = \rho bd = (0.0107)(12\text{IN})(21.5\text{ IN.}) = 2.761\text{ IN.}^2$

$R_{u\,(REVISED)} = \frac{M_n}{bd^2} = \frac{(278.784\text{ K-FT})(12000\text{ LB-IN/K-FT})}{(12\text{ IN.})(21.5\text{ IN})^2} = 603$ PSI

$\rho_{REVISED} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_u}{f_Y}}\right) = \frac{1}{17.647}\left(1 - \sqrt{1 - \frac{(2)(17.647)(603)}{60000}}\right) = 0.0111$

$A_s = \rho bd = (0.0111)(12\text{ IN.})(21.5\text{ IN.}) = 2.876\text{ IN.}^2$

CHECK SHEAR:

$V_u = \phi V_n$
$V_n = V_c + V_s$

$$V_{u\,(MAX)} = \frac{1.15\omega L}{2} = \frac{(1.15)(4.356\text{ KLF})(24\text{ FT})}{2} = 60.113^K$$

$$V_c = 2\sqrt{f'_c}\,bd = (2)(\sqrt{4000\text{ PSI}})(12\text{ IN})(21.5\text{ IN.}) = 32.635^K$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{60.113^K}{0.85} - 32.635^K = 38.086^K$$

$$4\sqrt{f'_c}\,b_w d = (4)(\sqrt{4000\text{ PSI}})(12\text{ IN.})(21.5\text{ IN.}) = 65.269^K > V_s \;\therefore\; S_{MAX} = \frac{d}{2} = 10.750\text{ IN.}$$

$$V_s = \frac{A_v f_y d}{s} \;\therefore\; s = \frac{A_v f_y d}{V_s} = \frac{(A_v)(40\text{ KSI})(21.5\text{ IN.})}{38.086^K} = 22.580\,A_v$$

STIRRUP SIZE ⟶ #3 @ 4.968 IN. O.C.
#4 @ 9.032 IN. O.C.
#5 @ 14.000 IN. O.C. ⟵ N.G. > $d/2$

USE 12"x24" INTERIOR BEAMS (FLOORS)

END BEAMS (FLOORS)

END BEAMS 32'-0" SPAN

ASSUME DEPTH = H = 24 IN. (d = 21 IN.)

LOADS: 1) UNIFORM LOAD DUE TO DEAD LOAD
2) UNIFORM LOAD DUE TO LIVE LOAD

LOAD 1)
$$W_{SLAB} = (5.25/12\text{ FT})(0.150\text{ KCF})(6\text{ FT.}) = 0.394\text{ KLF}$$
$$W_{SELF\ WT.} = \text{ASSUME } 0.500\text{ KLF} = 0.500\text{ KLF}$$
$$W_{FACADE} = (14\text{ FT})(0.080\text{ KSF}) = 1.120\text{ KLF}$$
$$W_{DL} = 2.014\text{ KLF}$$

LOAD 2)
$$W_{LIVE} = (6\text{ FT})(0.080\text{ KSF}) = \quad W_{LL} = 0.480\text{ KLF}$$

$$W_u = (1.4)(2.014\text{ KLF}) + (1.7)(0.480\text{ KLF}) = 3.636\text{ KLF}$$

$$\text{MAXIMUM NEGATIVE MOMENT} = \frac{W_u L^2}{10} = \frac{(3.636\text{ KLF})(32\text{ FT})^2}{10} = 372.326\text{ K-FT.}$$
$$\text{MAXIMUM POSITIVE MOMENT} = \frac{W_u L^2}{14} = \frac{(3.636\text{ KLF})(32\text{ FT})^2}{14} = 265.947\text{ K-FT.}$$

$\rho_{DESIRED} = 0.0107$
$m = 17.647$
$R_u = 581$ PSI

$$M_n = \frac{M_u}{\phi} = \frac{372.326\text{ K-FT}}{0.9} = 413.696\text{ K-FT}$$

$$\text{REQ'D } bd^2 = \frac{M_n}{R_u} = \frac{(413.696\text{ K-FT})(12000\text{ LB-IN/K-FT})}{581\text{ PSI}} = 8544.487\text{ IN.}^3$$

$d = 21$ IN. $\therefore$ $b = 19.375$ TRY $b = 18$ IN.

$$A_s = \rho bd = (0.0107)(18\text{ IN.})(21\text{ IN.}) = 4.045\text{ IN.}^2$$

$$R_{u\,(REVISED)} = \frac{M_n}{bd^2} = \frac{(413.696\text{ K-FT})(12000\text{ LB-IN/K-FT})}{(18\text{ IN})(21\text{ IN})^2} = 625\text{ PSI}$$

$$\rho_{REVISED} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_u}{f_y}}\right) = \frac{1}{17.647}\left(1 - \sqrt{1 - \frac{(2)(17.647)(625)}{60000}}\right) = 0.0116$$

$$A_s = \rho bd = (0.0116)(18\text{ IN.})(21\text{ IN.}) = 4.385\text{ IN.}^2$$

CHECK SHEAR:

$V_u = \phi V_n$
$V_n = V_c + V_s$

$$V_{u(MAX)} = \frac{1.15wL}{2} = \frac{(1.15)(3.636\ KLF)(32\ FT)}{2} = 66.902^K$$

$$V_c = 2\sqrt{f'_c}\,bd = (2)(\sqrt{4000\ PSI})(18\ IN.)(21\ IN.) = 47.814^K$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{66.902^K}{0.85} - 47.814^K = 30.894^K$$

$$4\sqrt{f'_c}\,bwd = (4)(\sqrt{4000\ PSI})(18\ IN.)(21\ IN.) = 95.627^K > V_s \quad \therefore S_{max} = \frac{d}{2} = 10.5\ IN.$$

$$V_s = \frac{A_v f_y d}{S} \quad \therefore \quad S = \frac{A_v f_y d}{V_s} = \frac{(A_v)(40\ KSI)(21\ IN.)}{30.894^K} = 27.190\ A_v$$

STIRRUP SIZE $\longrightarrow$ #3 @ 5.982 IN. O.C.
#4 @ 10.876 IN. O.C. $\longleftarrow$ N.G. $> d/2$

USE 18" x 24" END BEAMS (FLOORS)

⌗ INTERMEDIATE BEAMS (ROOF)

⌗ INTERMEDIATE BEAMS 32'-0" SPAN

ASSUME DEPTH = H = 24 IN. (d = 21 IN.)

LOADS: 1) UNIFORM LOAD DUE TO DEAD LOAD
2) UNIFORM LOAD DUE TO LIVE LOAD

LOAD 1) $W_{SLAB} = (5.25/12\text{ FT})(0.150\text{ KCF})(12\text{ FT}) = 0.788$ KLF
W_{SELFWT} = ASSUME 0.300 KLF $= 0.300$ KLF
$W_{DL} = 1.088$ KLF

LOAD 2) $W_{LIVE} = (12\text{ FT})(0.030\text{ KSF}) = W_{LL} = 0.360$ KLF

$W_u = (1.4)(1.088\text{ KLF}) + (1.7)(0.360\text{ KLF}) = 2.135$ KLF

MAXIMUM NEGATIVE MOMENT $= \dfrac{WL^2}{10} = \dfrac{(2.135\text{ KLF})(32\text{ FT})^2}{10} = 218.624$ K-FT.

MAXIMUM POSITIVE MOMENT $= \dfrac{WL^2}{14} = \dfrac{(2.135\text{ KLF})(32\text{ FT})^2}{14} = 156.160$ K-FT.

$\rho_{DESIRED} = 0.0107$
$m = 17.647$
$R_u = 581$ PSI

$$M_n = \frac{M_u}{\phi} = \frac{(218.624\text{ K-FT})}{(0.9)} = 242.916\text{ K-FT.}$$

$$\text{REQ'D } bd^2 = \frac{M_n}{R_u} = \frac{(242.916\text{ K-FT})(12000\text{ LB-IN/K-FT})}{581\text{ PSI}} = 5017.189\text{ IN}^3$$

d = 21 IN. ∴ b = 11.377 IN. TRY b = 12 IN.

$$A_s = \rho bd = (0.0107)(12\text{ IN.})(21\text{ IN.}) = 2.696\text{ IN}^2$$

$$R_{u\,(REVISED)} = \frac{M_n}{bd^2} = \frac{(242.916\text{ K-FT})(12000\text{ LB-IN/K-FT})}{(12\text{ IN})(21\text{ IN})^2} = 551\text{ PSI}$$

$$\rho_{REVISED} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_u}{f_y}}\right) = \frac{1}{17.647}\left(1 - \sqrt{1 - \frac{(2)(17.647)(551)}{60000}}\right) = 0.0101$$

$$A_s = \rho bd = (0.0101)(12\text{ IN.})(21\text{ IN.}) = 2.545\text{ IN}^2$$

CHECK SHEAR

$V_u = \phi V_n$
$V_n = V_c + V_s$

$$V_{u(MAX)} = \frac{1.15wL}{2} = \frac{(1.15)(2.135\text{ KLF})(32\text{ FT})}{2} = 39.284^k$$

$$V_c = 2\sqrt{f'_c}\,bd = (2)(\sqrt{4000\text{ PSI}})(12\text{ IN})(21\text{ IN}) = 31.876^k$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{39.284^k}{0.85} - 31.876^k = 14.340^k$$

$$4\sqrt{f'_c}\,b_w d = (4)(\sqrt{4000\text{ PSI}})(12\text{ IN.})(21\text{ IN.}) = 63.752^k > V_s \;\therefore\; S_{max} = \frac{d}{2} = 10.5\text{ IN.}$$

$$V_s = \frac{A_v f_y d}{s} \;\therefore\; s = \frac{A_v f_y d}{V_s} = \frac{(A_v)(40\text{ KSI})(21\text{ IN.})}{14.340^k} = 58.577\,A_v$$

STIRRUP SIZE ⟶ #3 @ 12.887 IN. O.C. ⟵ N.G. > d/2

USE 12"x24" INTERMEDIATE BEAMS (ROOF)

EXTERIOR BEAMS (ROOF)

EXTERIOR BEAMS 24'-0" SPAN

ASSUME DEPTH = H = 24 IN. (d = 21.5 IN.)

LOADS: 1) CONCENTRATED LOAD AT MIDSPAN FROM INTERMEDIATE BEAMS
2) UNIFORM LOAD FROM FACADE
3) UNIFORM LOAD FROM SELF WEIGHT

LOAD 1) $\frac{PL}{8} = \frac{wL_n^2}{10} \therefore w = \frac{10PL}{8L_n^2}$

CONCENTRATED LOAD $P = (16\text{ FT})(2.135\text{ KLF}) = 34.160^k$

EQUIVALENT LOAD / FT $= \frac{(10)(34.160^k)(24\text{ FT})}{(8)(24\text{ FT})^2} = 1.779\text{ KLF}$

LOAD 2) UNIFORM LOAD FROM FACADE = (1.4)(0.338 KLF) = 0.473 KLF

LOAD 3) UNIFORM LOAD · SELF WT. (ASSUMED) = (1.4)(0.300 KLF) = 0.420 KLF

$w_u = 2.672\text{ KLF}$

MAXIMUM NEGATIVE MOMENT $= \frac{wL^2}{10} = \frac{(2.672\text{ KLF})(24\text{ FT})^2}{10} = 153.917\text{ K-FT.}$

MAXIMUM POSITIVE MOMENT $= \frac{wL^2}{14} = \frac{(2.672\text{ KLF})(24\text{ FT})^2}{14} = 109.934\text{ K-FT.}$

$\rho_{DESIRED} = 0.0107$
$m = 17.647$
$R_u = 581\text{ PSI}$

$M_n = \frac{M_u}{\phi} = \frac{(153.917\text{ K-FT})}{(0.9)} = 171.019\text{ K-FT.}$

$\text{REQ'D } bd^2 = \frac{M_n}{R_u} = \frac{(171.019\text{ K-FT})(12000\text{ LB-IN/K-FT})}{581\text{ PSI}} = 3532.232\text{ IN.}^3$

$d = 21.5$ IN. $\therefore b = 7.641$ IN. TRY $b = 12$ IN. (CONSTRUCTION MIN.)

$A_s = \rho bd = (0.0107)(12\text{ IN})(21.5\text{ IN}) = 2.761\text{ IN.}^2$

$R_{u\,(REVISED)} = \frac{M_n}{bd^2} = \frac{(171.019\text{ K-FT})(12000\text{ LB-IN/K-FT})}{(12\text{ IN})(21.5\text{ IN})^2} = 370\text{ PSI}$

$\rho_{REVISED} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{(2)(m)(R_u)}{f_y}}\right) = \frac{1}{17.647}\left(1 - \sqrt{1 - \frac{(2)(17.647)(370)}{60000}}\right) = 0.0065$

$A_s = \rho bd = (0.0065)(12\text{ IN})(21.5\text{ IN.}) = 1.677\text{ IN.}^2$

CHECK SHEAR:

$V_u = \phi V_n$
$V_n = V_c + V_s$

$$V_{u(max)} = \frac{1.15 wL}{2} = \frac{(1.15)(2.672\ KLF)(24\ FT)}{2} = 36.874^K$$

$$V_c = 2\sqrt{f'_c}\, bd = (2)(\sqrt{4000\ PSI})(12\ IN.)(21.5\ IN.) = 32.635^K$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{36.874^K}{0.85} - 32.635^K = 10.746^K$$

$$4\sqrt{f'_c}\, b_w d = (4)(\sqrt{4000\ PSI})(12\ IN)(21.5\ IN.) = 65.269^K > V_s \therefore S_{max} = \frac{d}{2} = 10.75\ IN.$$

$$V_s = \frac{A_v f_y d}{s} \therefore s = \frac{A_v f_y d}{V_s} = \frac{(A_v)(40\ KSI)(21.5\ IN)}{10.746^K} = 80.030 A_v$$

STIRRUP SIZE $\longrightarrow$ #3 @ 17.607 IN. O.C. $\longleftarrow$ N.G. $> d/2$

USE 12"x 24" EXTERIOR BEAMS (ROOF)

INTERIOR BEAMS (ROOF)

INTERIOR BEAMS

ASSUME DEPTH = H = 24 IN. (d = 21.5 IN.)

LOADS: 1) CONCENTRATED LOAD AT MIDSPAN FROM INTERMEDIATE BEAMS
2) UNIFORM LOAD FROM SELF WEIGHT

LOAD 1) $\frac{PL}{8} = \frac{WL^2}{10} \therefore W = \frac{10PL}{8L_n^2}$

CONCENTRATED LOAD $P = (22\text{FT})(2.135\text{KLF}) = 46.920^K$

EQUIVALENT LOAD $= \frac{(10)(46.920)(24\text{FT})}{(8)(24\text{FT.})^2} = 2.446$ KLF

LOAD 2) UNIFORM LOAD - SELF WT. (ASSUMED) $= (0.300\text{KLF})(1.4) = 0.420$ KLF

$W_u = 2.866$ KLF

MAXIMUM NEGATIVE MOMENT $= \frac{WL^2}{10} = \frac{(2.866\text{ KLF})(24\text{FT})^2}{10} = 165.082$ K-FT.

MAXIMUM POSITIVE MOMENT $= \frac{WL^2}{14} = \frac{(2.866\text{ KLF})(24\text{FT})^2}{14} = 117.915$ K-FT.

$\rho_{DESIRED} = 0.0107$
$m = 17.647$
$R_u = 581$ PSI

$M_n = \frac{M_u}{\phi} = \frac{(165.082\text{ K-FT})}{(0.9)} = 183.424$ K-FT.

REQ'D $bd^2 = \frac{M_n}{R_u} = \frac{(183.424\text{ K-FT})(12000\text{ LB}\cdot\text{IN/K-FT})}{581\text{ PSI}} = 3788.457\text{ IN}^3$

$d = 21.5$ IN. $\therefore b = 8.196$ TRY $b = 12$ IN. (CONSTRUCTION MIN.)

$A_s = \rho bd = (0.0107)(12\text{ IN})(21.5\text{ IN.}) = 2.761\text{ IN}^2$

$R_{u(REVISED)} = \frac{M_n}{bd^2} = \frac{(183.424\text{ K-FT})(12000\text{ LB}\cdot\text{IN/K-FT})}{(12\text{IN})(21.5\text{ IN})^2} = 397$ PSI

$\rho_{REVISED} = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2m R_u}{F_Y}}\right) = \frac{1}{17.647}\left(1 - \sqrt{1 - \frac{(2)(17.647)(397)}{60000}}\right) = 0.0071$

$A_s = \rho bd = (0.0071)(12\text{ IN.})(21.5\text{ IN.}) = 1.820\text{ IN.}^2$

CHECK SHEAR:

$V_u = \phi V_n$
$V_n = V_c + V_s$

$$V_{u(MAX)} = \frac{1.15wL}{2} = \frac{(1.15)(2.866\text{ KLF})(24\text{ FT})}{2} = 39.551^K$$

$$V_c = 2\sqrt{f'c}\,bd = (2)(\sqrt{4000\text{ PSI}})(12\text{ IN})(21.5\text{ IN}) = 32.635^K$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{39.551^K}{0.85} - 32.635^K = 11.311^K$$

$$4\sqrt{f'c}\,b_w d = (4)(\sqrt{4000\text{ PSI}})(12\text{ IN})(21.5\text{ IN}) = 65.269^K > V_s \therefore S_{MAX} = \frac{d}{2} = 10.75\text{ IN}$$

$$V_s = \frac{A_v F_y d}{S} \quad \therefore S = \frac{A_v F_y d}{V_s} = \frac{(A_v)(40\text{ KSI})(21.5\text{ IN})}{11.311^K} = 76.032\,A_v$$

STIRRUP SIZE $\longrightarrow$ #3 @ 16.727 IN. O.C. $\leftarrow$ N.G. > d/2

USE 12" x 24" INTERIOR BEAMS (ROOF)

END BEAMS (ROOF)

END BEAMS 32'-0" SPAN

ASSUME DEPTH = H = 24 IN. (d = 21 IN.)

LOADS: 1.) UNIFORM LOAD DUE TO DEAD LOAD
2.) UNIFORM LOAD DUE TO LIVE LOAD

LOAD 1) W_{SLAB} = (5.25/12 FT)(0.150 KCF)(6 FT.) = 0.394 KLF
W_{SELFWT} = ASSUME 0.500 KLF = 0.300 KLF
W_{FACADE} = (7.5 FT)(0.080 KSF) = 0.600 KLF
W_D = 1.294 KLF

LOAD 2) W_{LIVE} = (6 FT)(0.030 KSF) = W_L = 0.180 KLF

W_u = (1.4)(1.294 KLF) + (1.7)(0.180 KLF) = 2.118 KLF

MAXIMUM NEGATIVE MOMENT = $\frac{WL^2}{10} = \frac{(2.118\text{ KLF})(32\text{ FT})^2}{10}$ = 216.883 K-FT.

MAXIMUM POSITIVE MOMENT = $\frac{WL^2}{14} = \frac{(2.118\text{ KLF})(32\text{ FT})^2}{14}$ = 154.917 K-FT.

$\rho_{DESIRED}$ = 0.0107
m = 17.647
R_u = 581 PSI

$M_n = \frac{M_u}{\phi} = \frac{(216.883\text{ K-FT.})}{0.9}$ = 240.981 K-FT

REQ'D $bd^2 = \frac{M_n}{R_u} = \frac{(240.981\text{ K-FT})(12000\text{ LB-IN/K-FT})}{581\text{ PSI}}$ = 4977.235 $IN.^3$

d = 21 IN. ∴ b = 11.286 IN   TRY b = 12 IN.

$A_s = \rho bd$ = (0.0107)(12 IN.)(21 IN.) = 2.696 $IN.^2$

$R_{u(REVISED)} = \frac{M_n}{bd^2} = \frac{(240.981\text{ K-FT})(12000\text{ LB-IN/K-FT})}{(12\text{ IN.})(21\text{ IN})^2}$ = 546 PSI

$\rho_{REVISED} = \frac{1}{m}\left(1-\sqrt{1-\frac{(2)(m)(R_u)}{f_y}}\right) = \frac{1}{17.647}\left(1-\sqrt{1-\frac{(2)(17.647)(546)}{60000}}\right)$ = 0.0100

$A_s = \rho bd$ = (0.0100)(12 IN.)(21 IN.) = 2.515 $IN.^2$

CHECK SHEAR:

$V_u = \phi V_n$
$V_n = V_c + V_s$

$$V_{u(max)} = \frac{1.15wL}{2} = \frac{(1.15)(2.118\ KLF)(32\ FT)}{2} = 38.971^K$$

$$V_c = 2\sqrt{f'_c}\,bd = (2)(\sqrt{4000\ PSI})(12\ IN)(21\ IN.) = 31.876^K$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{38.971^K}{0.85} - 31.876^K = 13.972^K$$

$$4\sqrt{f'_c}\,b_w d = (4)(\sqrt{4000\ PSI})(12\ IN)(21\ IN) = 63.752^K > V_s \therefore S_{max} = \frac{d}{2} = 10.5\ IN.$$

$$V_s = \frac{A_v F_{y} d}{S} \quad \therefore S = \frac{A_v F_y d}{V_s} = \frac{(A_v)(40\ KSI)(21\ IN)}{13.972^K} = 60.120\ A_v$$

STIRRUP SIZE $\longrightarrow$ #3 @ 13.226 IN. O.C $\longleftarrow$ N.G $> d/2$

USE 12"x24" END BEAMS (20F)

COLUMN DESIGN

THE PRELIMINARY SIZE OF ALL COLUMNS HAS BEEN ESTIMATED USING A "RULE OF THUMB" PROCEDURE TAUGHT TO ME IN ARCH. 6100 - STRUCTURAL SYSTEMS.

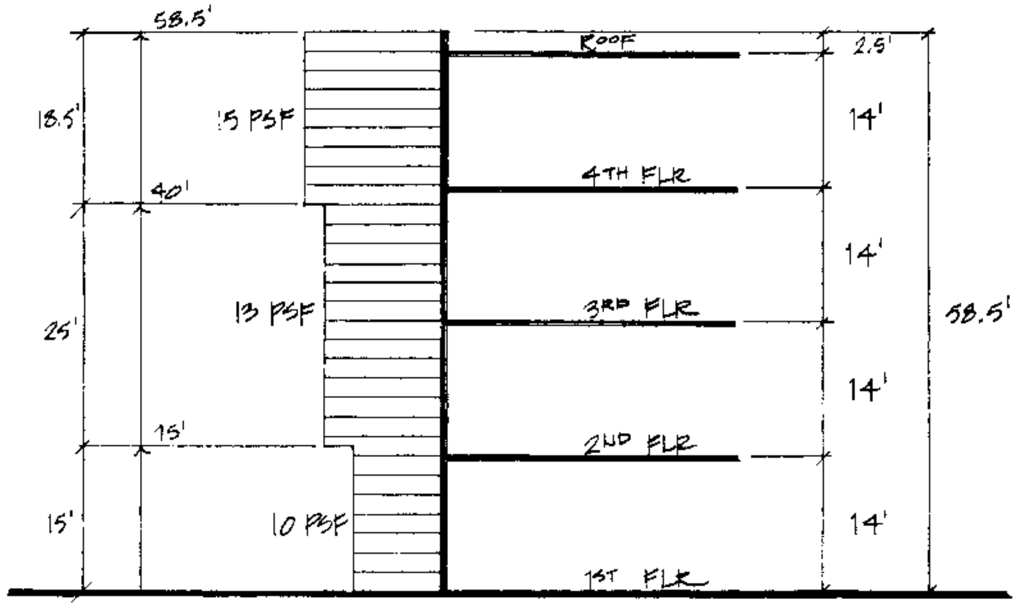
"RULE OF THUMB" FOR CONC. COLUMNS → SIZE = 16"+1" PER STORY FOR MULTI-STORY CONSTRUCTION.

COLUMNS OF THIS SIZE SHOULD ALLOW ENOUGH ROOM FOR STEEL REINFORCEMENT AT JOINTS.

☐ WIND ANALYSIS

DESIGN VELOCITY = 80 MPH

FLOOR #	ELEVATION (FT.)	PRESSURE (PSF)
2	14	11.29
3	28	13.00
4	42	14.29
R	56	15.00



WIND ANALYSIS

WIND LOADS

FRAMES 1-1 & 9-9

WINDWARD :	ROOF	(9.5')(12')(15 PSF)(.8)	= 1368.0 LB
	4TH	(5')(12')(13 PSF)(.8) + (2'+7')(12')(15 PSF)(.8)	= 1920.0
	3RD	(7'+7')(12')(13 PSF)(.8)	= 1747.2
	2ND	(7'+1')(12')(10 PSF)(.8) + (6')(12')(13 PSF)(.8)	= 1516.8
LEEWARD :	ROOF	(9.5')(12')(15 PSF)(.5)	= 855.0 LB
	4TH	(5')(12')(13 PSF)(.5) + (2'+7')(12')(15 PSF)(.5)	= 1200.0
	3RD	(7'+7')(12')(13 PSF)(.5)	= 1092.0
	2ND	(7'+1')(12')(10 PSF)(.5) + (6')(12')(13 PSF)(.5)	= 948.0

TOTAL LOAD:	ROOF	1368.0 + 855.0 = 2.223 K
	4TH	1920.0 + 1200.0 = 3.120 K
	3RD	1747.2 + 1092.0 = 2.839 K
	2ND	1516.8 + 948.0 = 2.465 K

FRAMES 2-2, 5-5, & 8-8

WINDWARD :	ROOF	(9.5')(24')(15 PSF)(.8)	= 2736.0 LB
	4TH	(5')(24')(13 PSF)(.8) + (2'+7')(24')(15 PSF)(.8)	= 3840.0
	3RD	(7'+7')(24')(13 PSF)(.8)	= 3494.0
	2ND	(7'+1')(24')(10 PSF)(.8) + (6')(24')(13 PSF)(.8)	= 3034.0
LEEWARD :	ROOF	(9.5')(24')(15 PSF)(.5)	= 1710.0 LB
	4TH	(5')(24')(13 PSF)(.5) + (2'+7')(24')(15 PSF)(.5)	= 2400.0
	3RD	(7'+7')(24')(13 PSF)(.5)	= 2184.0
	2ND	(7'+1')(24')(10 PSF)(.5) + (6')(24')(13 PSF)(.5)	= 1896.0

TOTAL LOAD:	ROOF	2736.0 + 1710.0 = 4.446 K
	4TH	3840.0 + 2400.0 = 6.240 K
	3RD	3494.0 + 2184.0 = 5.678 K
	2ND	3034.0 + 1896.0 = 4.930 K

FRAMES 3-3, 4-4, 6-6, & 7-7

WINDWARD :	ROOF	(9.5')(18')(15 PSF)(.8)	= 2052.0 LB
	4TH	(5')(18')(13 PSF)(.8) + (2'+7')(18')(15 PSF)(.8)	= 2880.0
	3RD	(7'+7')(18')(13 PSF)(.8)	= 2620.8
	2ND	(7'+1')(18')(10 PSF)(.8) + (6')(18')(13 PSF)(.8)	= 2275.2
LEEWARD :	ROOF	(9.5')(18')(15 PSF)(.5)	= 1282.5 LB
	4TH	(5')(18')(13 PSF)(.5) + (2'+7')(18')(15 PSF)(.5)	= 1800.0
	3RD	(7'+7')(18')(13 PSF)(.5)	= 1638.0
	2ND	(7'+1')(18')(10 PSF)(.5) + (6')(18')(13 PSF)(.5)	= 1188.0

☐ WIND ANALYSIS

☐ WIND LOADS

☐ FRAMES 3-3, 4-4, 6-6, 7-7

TOTAL LOAD:	ROOF	2052.0 + 1282.5	= 3.335 K
	4TH	2880.0 + 1800.0	= 4.680 K
	3RD	2620.0 + 1638.0	= 4.259 K
	2ND	2275.2 + 1188.0	= 3.463 K

☐ FRAMES A-A & D-D

WINDWARD:	ROOF	(9.5')(16')(15 PSF)(.8)	= 1824.0 LB
	4TH	(5')(16')(13 PSF)(.8) + (2'+7')(16')(15 PSF)(.8)	= 2560.0
	3RD	(7'+7')(16')(13 PSF)(.8)	= 2329.6
	2ND	(7'+1')(16')(10 PSF)(.8) + (6')(16')(13 PSF)(.8)	= 2022.4
LEEWARD:	ROOF	(9.5')(16')(15 PSF)(.5)	= 1140.0 LB
	4TH	(5')(16')(13 PSF)(.5) + (2'+7')(16')(15 PSF)(.5)	= 1600.0
	3RD	(7'+7')(16')(13 PSF)(.5)	= 1456.0
	2ND	(7'+1')(16')(10 PSF)(.5) + (6')(16')(13 PSF)(.5)	= 1264.0
TOTAL LOAD:	ROOF	1824.0 + 1140.0	= 2.964 K
	4TH	2560.0 + 1600.0	= 4.160 K
	3RD	2329.6 + 1456.0	= 3.786 K
	2ND	2022.4 + 1264.0	= 3.286 K

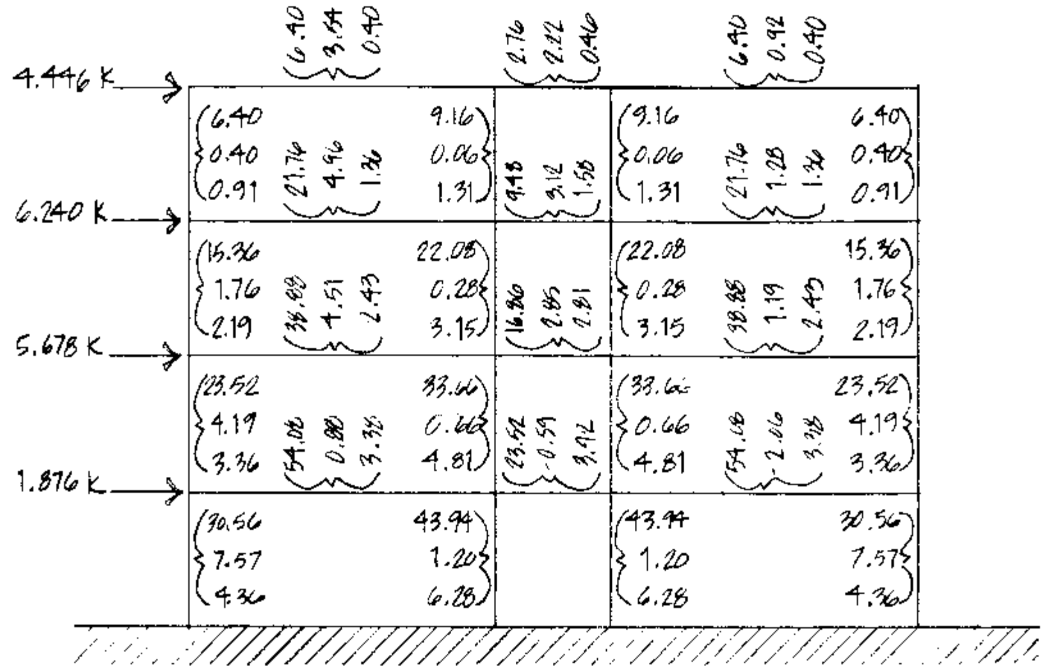
☐ FRAMES B-B & C-C

WINDWARD:	ROOF	(9.5')(22')(15 PSF)(.8)	= 2508.0 LB
	4TH	(5')(22')(13 PSF)(.8) + (2'+7')(22')(15 PSF)(.8)	= 3520.0
	3RD	(7'+7')(22')(13 PSF)(.8)	= 3203.2
	2ND	(7'+1')(22')(10 PSF)(.8) + (6')(22')(13 PSF)(.8)	= 2780.8
LEEWARD:	ROOF	(9.5')(22')(15 PSF)(.5)	= 1567.5 LB
	4TH	(5')(22')(13 PSF)(.5) + (2'+7')(22')(15 PSF)(.5)	= 2200.0
	3RD	(7'+7')(22')(13 PSF)(.5)	= 2002.0
	2ND	(7'+1')(22')(10 PSF)(.5) + (6')(22')(13 PSF)(.5)	= 1738.0
TOTAL LOAD:	ROOF	2508.0 + 1567.5	= 4.076 K
	4TH	3520.0 + 2200.0	= 5.720 K
	3RD	3203.2 + 2002.0	= 5.205 K
	2ND	2780.8 + 1738.0	= 4.519 K

☐ WIND ANALYSIS

☐ CANTILEVER WIND ANALYSIS – FRAMES 2-2, 5-5, 8-8

MOMENT K-FT
AXIAL LOAD KIPS
SHEAR KIPS



SEISMIC LOADS

$V = ZKCW$ — V = THE TOTAL LATERAL FORCE OR SHEAR AT THE BASE.
Z = ZONE COEFFICIENT
C = COEFFICIENT FOR BASE SHEAR
W = DEAD LOAD OF BUILDING
K = NUMERICAL COEFFICIENT

W (FLOOR):

$\left(\frac{5.25''}{12}\right)(150)(168')(76')$	837,900 LB	SLAB
$\left(\frac{18\times24}{144}\right)(150)(13)(76)$	444,600	INTERMEDIATE BEAMS
$\left(\frac{12\times24}{144}\right)(150)(2)(168)$	100,800	INTERIOR BEAMS
$\left(\frac{12\times24}{144}\right)(150)(2)(168)$	100,800	EXTERIOR BEAMS
$\left(\frac{18\times24}{144}\right)(150)(2)(76)$	68,400	END BEAMS
$\left(\frac{14\times14}{144}\right)(150)(36)(14-5.25/12)$	99,684	COLUMNS (ASSUMED)
TOTAL WEIGHT PER FLOOR	1,652,184 LB	

W (ROOF):

$\left(\frac{5.25}{12}\right)(150)(168)(76)$	837,900 LB	SLAB
$\left(\frac{14\times14}{144}\right)(150)(13)(76)$	345,800	INTERMEDIATE BEAMS
$\left(\frac{12\times24}{144}\right)(150)(2)(168)$	100,800	INTERIOR BEAMS
$\left(\frac{12\times24}{144}\right)(150)(2)(168)$	100,800	EXTERIOR BEAMS
$\left(\frac{12\times24}{144}\right)(150)(2)(76)$	45,600	END BEAMS
$\left(\frac{14\times14}{144}\right)(150)(36)(14-5.25/12)$	99,684	COLUMNS (ASSUMED)
TOTAL ROOF WEIGHT	1,530,584 LB	

TOTAL BUILDING WEIGHT (W) = 3(1652184 LB) + 1(1530584 LB) = 6,487,136 LB

$Z = 0.25$ FOR ZONE 1
$K = 0.67$ FOR DUCTILE FRAMES
$C = 0.05 \div \sqrt[3]{T}$ $\quad T = 0.10N = 0.4 \quad \therefore C = 0.067860$
$W = 6487136$ LB
$\therefore V = (0.25)(0.67)(0.067860)(6487136\text{ LB}) = 73736$ LB

SEISMIC LOADS

$$F_x = \frac{(V - F_t)\, W_x h_x}{\sum_{i=1}^{n} W_i h_i}$$

F_x = SEISMIC FORCE APPLIED AT HEIGHT X.
V = THE TOTAL LATERAL FORCE OR SHEAR AT BASE
$F_t = 0.004\, V (h_n / D_s)^2$
$W_x h_x$ = FLOOR (OR ROOF) WEIGHT × HEIGHT.

FOR LOADS PARALLEL TO GRID LINES A, B, C, D

$\Sigma W_i h_i$ = 1652184 LB (14'+28'+42') + 1530584 LB (56') = 224,496,160 FT-LB

$F_t = (0.004)(73736\text{ LB})(56' \div 168')^2 = 32.8$ LB

V = 73736 LB

FLOOR	LOAD ∥ TO A-A
ROOF	28.14 K
4TH	22.78 K
3RD	15.19 K
2ND	7.59 K

FOR LOADS PARALLEL TO GRID LINES 1 THRU 9

$\Sigma W_i h_i$ = 224,496,160 FT-LB

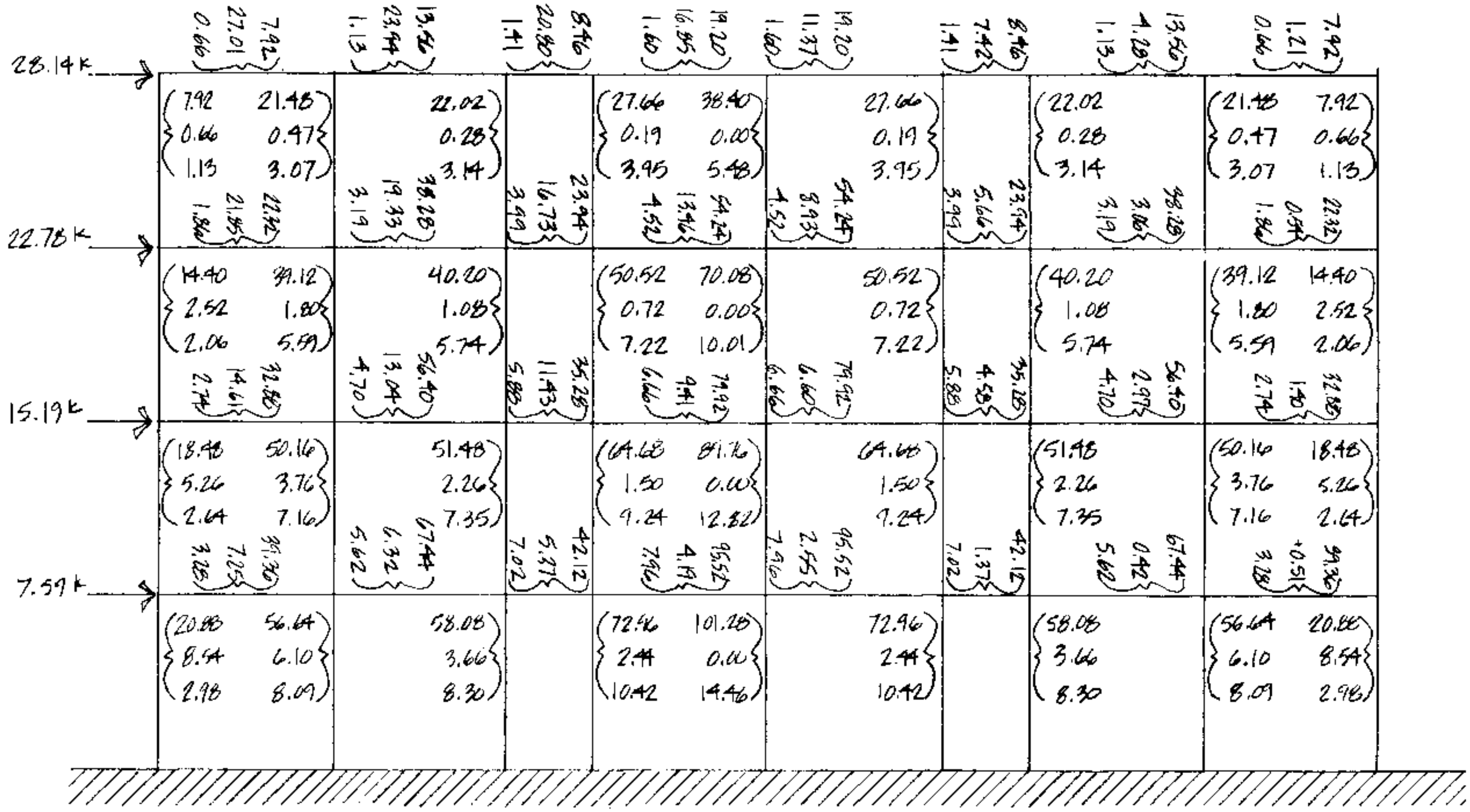
$F_t = (0.004)(73736\text{ LB})(56' \div 76')^2 = 160.1$ LB

V = 73736 LB

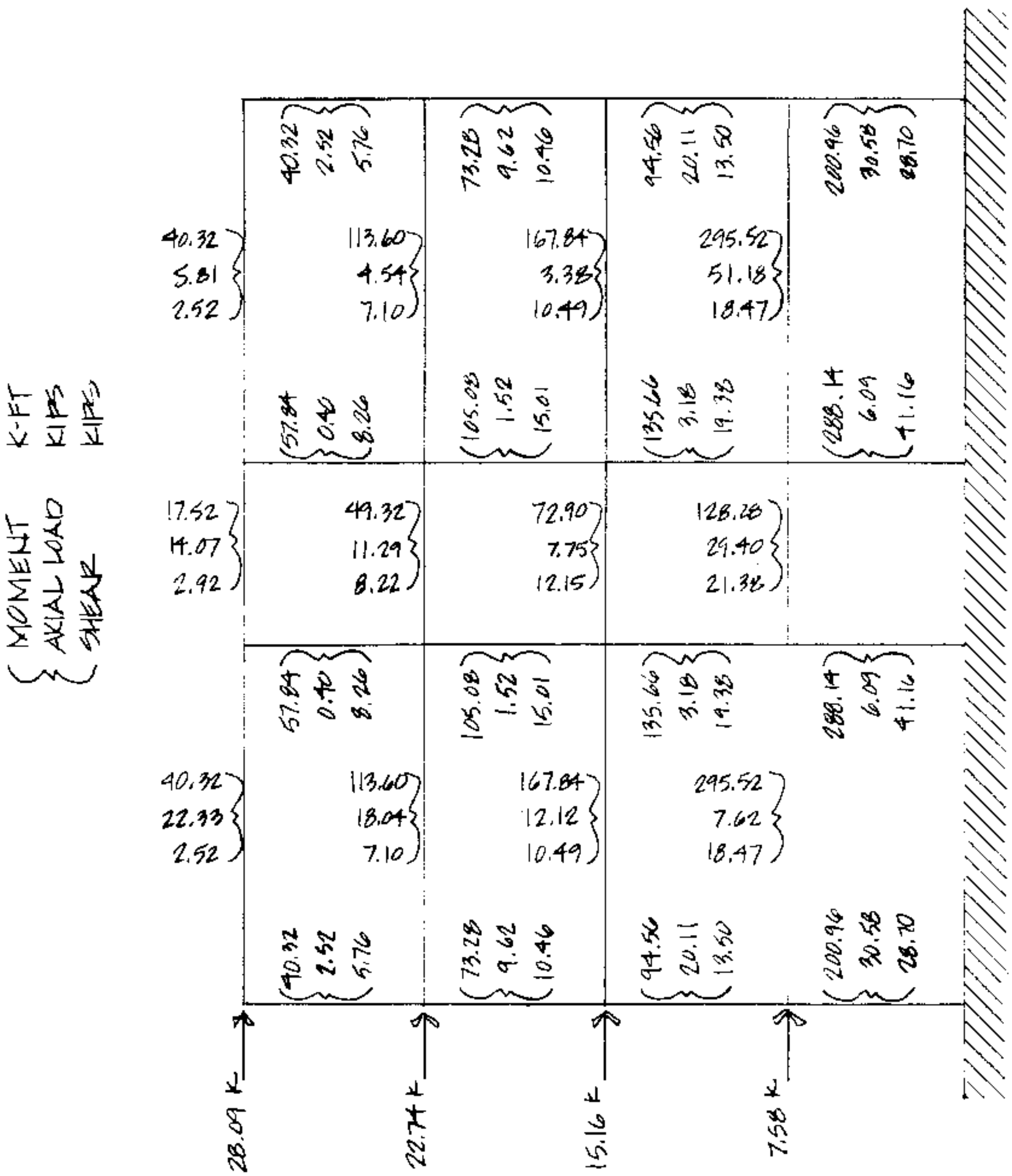
FLOOR	LOAD ∥ TO 5-5
ROOF	28.09 K
4TH	22.74 K
3RD	15.16 K
2ND	7.58 K

SEISMIC ANALYSIS FRAMES A,B,C,D

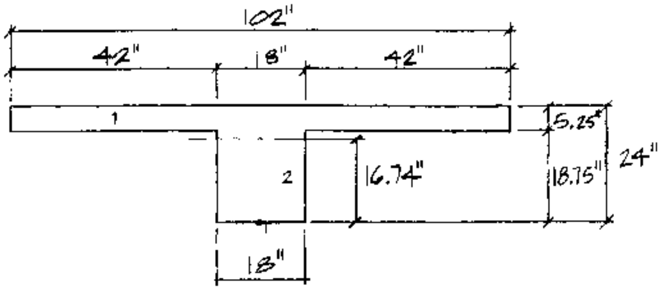
MOMENT K-FT.
AXIAL LOAD KIPS
SHEAR KIPS



SEISMIC ANALYSIS FRAMES 1 THRU 9



☐ EXTERIOR & INTERIOR - FLOOR BEAM



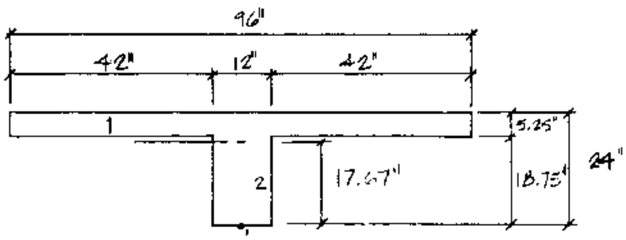
COMPONENT	AREA (IN.)	Y_m	$M_x = AY$
1	535.5	21.375	11446.31
2	337.5	9.375	3164.06
	873.0		14610.37

$\bar{Y} = M_x / A = 16.74$ IN.

COMPONENT	I_c	AREA	d	Ad^2	$I = I_c + Ad^2$
1	$bh^3/12 = 1229.98$	535.5	4.635	11504.27	12734.25
2	9887.70	337.5	7.365	18307.09	28194.79
					40929.04

SECTION PROPERTIES: $\boxed{A = 873 \text{ IN}^2}$ $\boxed{I = 40929 \text{ IN}^4}$ $\boxed{\bar{Y}_b = 16.59 \text{ IN}}$

☐ EXTERIOR & INTERIOR - ROOF BEAM



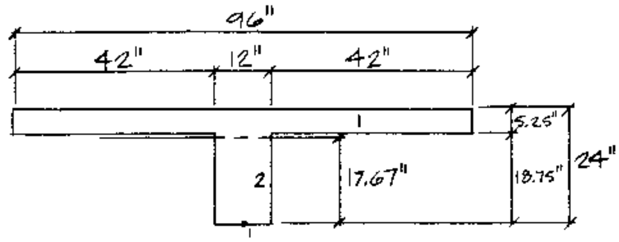
COMPONENT	AREA	Y_i	$M_x = AY_i$
1	504.0	21.375	10773.00
2	225.0	9.375	2109.38
	729.0		12882.38

$\bar{Y} = M_x / A = 17.67$ IN.

COMPONENT	I_c	AREA	d	Ad^2	$I = I_c + Ad^2$
1	1157.62	504.0	3.705	6918.42	8076.04
2	6591.80	225.0	8.295	15481.58	22073.38
					30149.42

SECTION PROPERTIES: $\boxed{A = 729 \text{ IN}^2}$ $\boxed{I = 30149 \text{ IN}^4}$ $\boxed{\bar{Y}_b = 17.67 \text{ IN}}$

☐ EXTERIOR & INTERIOR



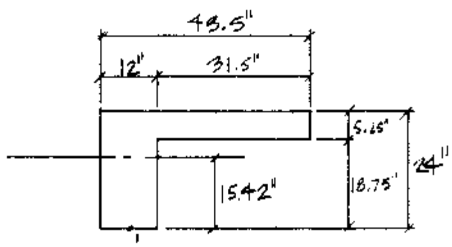
COMPONENT	AREA	Y_i	$M_x = AY_i$
1	504	21.375	10773.00
2	225	9.375	2109.38
	729		12882.38

$\bar{Y}_i = M_x / A = 17.67$ IN

COMPONENT	I_c	AREA	d		
1	1157.62	504	3.705	6918.42	8076.04
2	6591.80	225	8.295	15481.58	22073.38
					30149.42

SECTION PROPERTIES: $\boxed{A = 729 \text{ IN}^2}$ $\boxed{I = 30149 \text{ IN}^4}$ $\boxed{Y_b = 17.67 \text{ IN}}$

☐ EXTERIOR



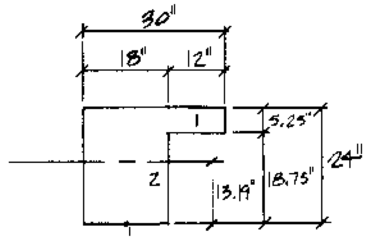
COMPONENT	AREA	Y_i	$M_x = AY_i$
1	228.38	21.375	4881.52
2	225.00	9.375	2109.38
	453.38		6990.90

$\bar{Y}_i = M_x / A = 15.42$ IN.

COMPONENT	I_c	AREA	d	Ad^2	$I = I_c + Ad^2$
1	524.55	228.38	5.955	8093.82	8623.37
2	6591.80	225.00	6.045	8221.96	14813.76
					23437.13

SECTION PROPERTIES: $\boxed{A = 453.38 \text{ IN}^2}$ $\boxed{I = 23437 \text{ IN}^4}$ $\boxed{Y_b = 15.42 \text{ IN.}}$

INTERIOR-FLOOR BEAM



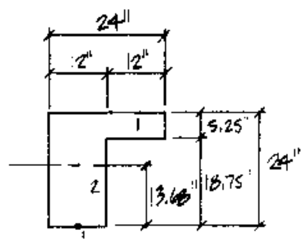
COMPONENT	AREA	Y_i	$M_x = AY_i$
1	157.50	21.375	3366.56
2	337.50	9.375	3164.06
	495.00		6530.62

$\bar{Y} = M_x / A = 13.19$ IN.

COMPONENT	I_c	AREA	d	Ad^2	$I = I_c + Ad^2$
1	361.76	157.50	8.185	10551.59	10913.35
2	9887.70	337.50	3.815	4912.05	14799.75
					25713.10

SECTION PROPERTIES: $\boxed{A = 495\ IN^2}$ $\boxed{I = 25713\ IN^4}$ $\boxed{Y_b = 13.19\ IN}$

INTERIOR-ROOF BEAM



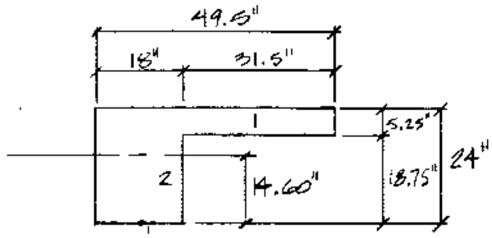
COMPONENT	AREA	Y_i	$M_x = AY_i$
1	126.00	21.375	2693.25
2	225.00	9.375	2109.38
	351.00		4802.63

$\bar{Y} = M_x / A = 13.68$ IN.

COMPONENT	I_c	AREA	d	Ad^2	$I = I_c + Ad^2$
1	289.41	126.00	7.695	7460.84	7750.25
2	6591.80	225.00	4.305	4169.93	10761.73
					18511.98

SECTION PROPERTIES: $\boxed{A = 351\ IN^2}$ $\boxed{I = 18512\ IN^4}$ $\boxed{Y_b = 13.68\ IN.}$

☐ EXTERIOR-FLOOR BEAM



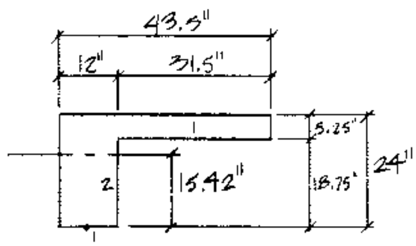
COMPONENT	AREA	Y_i	$M_x = AY_i$
1	259.88	21.375	5554.83
2	337.50	9.375	3164.06
	597.38		8718.89

$\bar{Y} = M_x/A = 14.60$ IN

COMPONENT	I_c	AREA	d	Ad^2	$I = I_c + Ad^2$
1	596.90	259.88	6.775	11928.65	12525.55
2	9887.70	337.50	5.225	9213.96	19101.66
					31627.21

SECTION PROPERTIES: $\boxed{A = 597.38 \text{ IN}^2}$ $\boxed{I = 31627 \text{ IN}^4}$ $\boxed{Y_b = 14.60 \text{ IN}}$

☐ EXTERIOR-ROOF BEAM



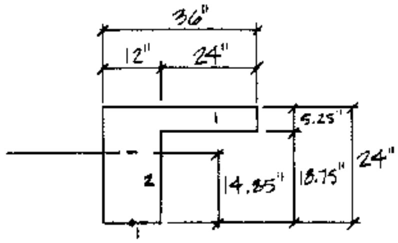
COMPONENT	AREA	Y_i	$M_x = AY_i$
1	228.38	21.375	4881.52
2	225.00	9.375	2109.38
	453.38		6990.90

$\bar{Y} = M_x/A = 15.42$ IN.

COMPONENT	I_c	AREA	d	Ad^2	$I = I_c + Ad^2$
1	524.55	228.38	5.955	8098.82	8623.37
2	6591.80	225.00	6.045	8221.96	14813.76
					23437.13

SECTION PROPERTIES: $\boxed{A = 453.38 \text{ IN}^2}$ $\boxed{I = 23437 \text{ IN}^4}$ $\boxed{Y_b = 15.42 \text{ IN}}$

☐ EXTERIOR & INTERIOR-FLOOR & ROOF BEAM



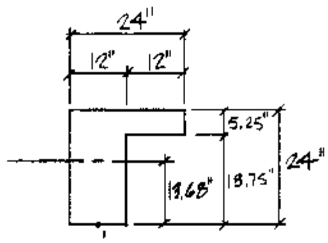
COMPONENT	AREA	Y_i	$M_x = AY_i$
1	189	21.375	4039.88
2	225	9.375	2109.38
	414		6149.26

$\bar{Y} = M_x / A = 14.85$ IN.

COMPONENT	I_c	AREA	d	Ad^2	$I = I_c + Ad^2$
1	434.11	189	6.525	8046.79	8480.90
2	6591.80	225	5.475	6744.52	13336.32
					21817.22

SECTION PROPERTIES: $\boxed{A = 414 \text{ IN}^2}$ $\boxed{I = 21817 \text{ IN}^4}$ $\boxed{Y_b = 14.85 \text{ IN.}}$

☐ INTERIOR - FLOOR & ROOF BEAM



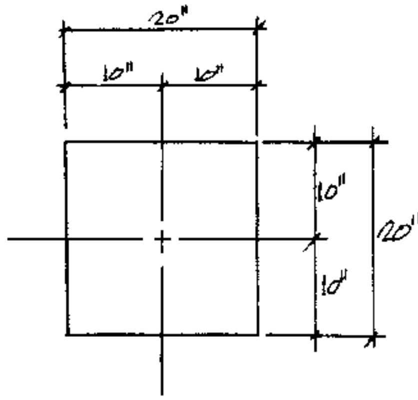
COMPONENT	AREA	Y_i	$M_x = AY_i$
1	186	21.375	2693.25
2	225	9.375	2109.38
	351		4802.63

$\bar{Y}_i = M_x / A = 13.68$ IN.

COMPONENT	I_c	AREA	d	Ad^2	$I = I_c + Ad^2$
1	289.41	186	7.695	7460.84	7750.25
2	6591.80	225	4.305	4169.93	10761.73
					18511.98

SECTION PROPERTIES : $\boxed{A = 351 \text{ IN}^2}$ $\boxed{I = 18512 \text{ IN}^4}$ $\boxed{Y_b = 13.68 \text{ IN}}$

EXTERIOR & INTERIOR COLUMNS



$A = bd = (20\ IN)^2 = 400\ IN^2$

$I = \frac{d^4}{12} = \frac{(20\ IN)^4}{12} = 13333.33$

SECTION PROPERTIES: $\boxed{A = 400\ IN.^2}$ $\boxed{I = 13333\ IN.^4}$ $\boxed{Y_b = 10\ IN.}$

BEAM LOADS FOR ANALYSIS

INTERMEDIATE BEAM LOADS

- SPAN = 32'-0"
- TRANSVERSE WIDTH = 12'-0"
- FLOORS
  - DEAD LOAD
    - SLAB (12 FT)(5.25 IN)(1FT/12IN)(.150 K/FT³) = 0.7875 K/FT
    - BEAM (18.75 IN)(18 IN)(1FT²/144 IN²)(.150 K/FT³) = 0.3516 K/FT
    - TOTAL 1.1391 K/FT
  - LIVE LOAD
    - UNIFORM (0.080 KSF)(12 FT) = 0.9600 K/FT
- ROOF
  - DEAD LOAD
    - SLAB (12FT)(5.25 IN)(1FT/12IN)(.150 K/FT³) = 0.7875 K/FT
    - BEAM (18.75 IN)(12 IN)(1FT²/144 IN²)(.150 K/FT³) = 0.2344 K/FT
    - TOTAL 1.0219 K/FT
  - LIVE LOAD
    - UNIFORM (0.030 KSF)(12 FT) = 0.3600 K/FT

INTERMEDIATE BEAM

- SPAN = 12'-0"
- TRANVERSE WIDTH = 6'-0"
- FLOOR
  - DEAD LOAD
    - SLAB (6FT)(5.25 IN)(1FT/12IN)(0.150 K/FT³) = 0.3938 K/FT
    - BEAM (18.75 IN)(18 IN)(1FT²/144 IN²)(.150 K/FT³) = 0.3516 K/FT
    - TOTAL 0.7454 K/FT
  - LIVE LOAD
    - UNIFORM (0.080 KSF)(6FT) = 0.4800 K/FT
- ROOF
  - DEAD LOAD
    - SLAB (6FT)(5.25 IN)(1FT/12IN)(0.150 K/FT³) = 0.3938 K/FT
    - BEAM (18.75 IN)(12 IN)(1FT²/144 IN²)(.150 K/FT³) = 0.2344 K/FT
    - TOTAL 0.6282 K/FT
  - LIVE LOAD
    - UNIFORM (0.030 KSF)(6FT) = 0.1800 K/FT

- INTERIOR GIRDERS
  - SPAN = 24'-0"
  - TRANSVERSE WIDTH = 6'-0"
  - FLOOR
    - DEAD LOAD
      - SLAB (6FT)(5.25 IN)(1FT/12 IN)(.150 K/FT³) = 0.3938 K/FT
      - BEAM (18.75 IN)(12 IN)(1FT²/144 IN²)(.150 K/FT³) = 0.2344 K/FT
      - TOTAL 0.6282 K/FT
      - CONC. LOAD @ MIDSPAN (16FT)(1.1391 K/FT)+(6FT)(.7454 K/FT) = 22.6980 K
    - LIVE LOAD
      - UNIFORM (0.080 KSF)(6FT) = 0.4800
      - CONC LOAD @ MIDSPAN (16FT)(.9600 K/FT) + (6FT)(0.4800 K/FT) = 18.2400 K
  - ROOF
    - DEAD LOAD
      - SLAB (6FT)(5.25 IN)(1FT/12 IN)(.150 K/FT³) = 0.3938 K/FT
      - BEAM (18.75 IN)(12 IN)(1FT²/144 IN²)(.150 K/FT³) = 0.2344 K/FT
      - TOTAL 0.6282 K/FT
      - CONC. LOAD @ MIDSPAN (16FT)(1.0219 K/FT)+(6FT)(0.6282 K/FT) = 20.1196 K
    - LIVE LOAD
      - UNIFORM (0.030 KSF)(6FT) = 0.1800 K/FT
      - CONC. LOAD @ MIDSPAN (16FT)(0.3600 K/FT)+(6FT)(.1800 K/FT) = 6.8400 K

- INTERIOR BEAM
  - SPAN = 12'-0"
  - TRANSVERSE WIDTH = 6'-0"
  - FLOOR
    - DEAD LOAD
      - SLAB (6FT)(5.25 IN)(1FT/12 IN)(.150 K/FT³) = 0.3938 K/FT
      - BEAM (18.75 IN)(12 IN)(FT²/144 IN²)(.150 K/FT³) = 0.2344 K/FT
      - TOTAL 0.6282 K/FT
    - LIVE LOAD
      - UNIFORM (0.080 KSF)(6FT) = 0.4800 K/FT

- ROOF
  - DEAD LOAD
    - SLAB (6FT)(5.25IN)(1FT/12IN)(.150 K/FT³) = 0.3938 K/FT
    - BEAM (18.75 IN)(12 IN)(1FT²/144 IN²)(.150 K/FT³) = 0.2344 K/FT
    - TOTAL 0.6282 K/FT
  - LIVE LOAD
    - UNIFORM (0.030 KSF)(6FT) = 0.1800 KLF
- EXTERIOR GIRDERS
  - SPAN = 24'-0"
  - TRANSVERSE 3'-6"
  - FLOOR
    - DEAD LOAD
      - SLAB (3.5FT)(5.25IN)(1FT/12IN)(.150 K/FT³) = 0.2297 K/FT
      - BEAM (18.75IN)(12IN)(1FT²/144IN²)(.150 K/FT³) = 0.2344 K/FT
      - FACADE (7FT)(0.080KSF)+(7FT)(0.005KSF) = 0.5950 K/FT
      - TOTAL 1.0591 K/FT
      - CONC LOAD (16FT)(1.1391 KLF) = 18.2256 K
        @ MIDSPAN
    - LIVE LOAD
      - UNIFORM (0.080KSF)(3FT) = 0.2400 K/FT
      - CONC LOAD (16FT)(0.9600 K/FT) = 15.3600 K
        @ MIDSPAN
  - ROOF
    - DEAD LOAD
      - SLAB (3.5FT)(5.25IN)(1FT/12IN)(.150 K/FT³) = 0.2297 K/FT
      - BEAM (18.75IN)(12IN)(1FT²/144IN²)(0.150K/FT³) = 0.2344 K/FT
      - FACADE (4FT)(0.080KSF)+(3.5FT)(0.005KSF) = 0.3375 K/FT
      - TOTAL 0.8016 K/FT
      - CONC LOAD (16FT)(1.0221 K/FT) = 16.3536 K
        @ MIDSPAN
    - LIVE LOAD
      - UNIFORM (0.030 KSF)(3FT) = 0.0900 K/FT
      - CONC. LOAD (16FT)(0.3600 K/FT) = 5.7600 K
        @ MIDSPAN

- EXTERIOR BEAM
  - SPAN = 12'-0"
  - TRANSVERSE WIDTH = 3'-6"
  - FLOOR
    - DEAD LOAD
      - SLAB (3.5 FT)(5.25 IN)(1 FT/12 IN)(.150 K/FT³) = 0.2297 K/FT
      - BEAM (18.75 IN)(12 IN)(1 FT²/144 IN²)(.150 K/FT³) = 0.2344 K/FT
      - FACADE (7 FT)(0.080 KSF)+(7 FT)(0.005 KSF) = 0.5950 K/FT
      - TOTAL 1.0591 K/FT
    - LIVE LOAD
      - UNIFORM (0.080 KSF)(3 FT) = 0.2400 K/FT
  - ROOF
    - DEAD LOAD
      - SLAB (3.5 FT)(5.25 IN)(1 FT/12 IN)(.150 K/FT³) = 0.2297 K/FT
      - BEAM (18.75 IN)(12 IN)(1 FT²/144 IN²)(.150 K/FT³) = 0.2344 K/FT
      - FACADE (4 FT)(0.080 KSF)+(3.5 FT)(0.005 KSF) = 0.3375 K/FT
      - TOTAL 0.8021 K/FT
    - LIVE LOAD
      - UNIFORM (0.030 KSF)(3 FT) = 0.0900 K/FT
- END BEAM
  - SPAN = 32'-0"
  - TRANSVERSE WIDTH = 6'-9"
  - FLOOR
    - DEAD LOAD
      - SLAB (6.75 FT)(5.25 IN)(1 FT/12 IN)(.150 K/FT³) = 0.4430 K/FT
      - BEAM (18.75 IN)(18 IN)(1 FT²/144 IN²)(.150 K/FT³) = 0.3516 K/FT
      - FACADE (14 FT)(0.080 KSF) = 1.1200 K/FT
      - TOTAL 1.9146 K/FT
    - LIVE LOAD
      - UNIFORM (0.080 KSF)(6.75 FT) = 0.5400 K/FT
  - ROOF
    - DEAD LOAD
      - SLAB (6.5 FT)(5.25 IN)(1 FT/12 IN)(.150 K/FT³) = 0.4266 K/FT
      - BEAM (18.75 IN)(12 IN)(1 FT²/144 IN²)(.150 K/FT³) = 0.2[illegible] K/FT
      - FACADE (7.5 FT)(0.080 KSF) = 0.6000 K/FT
      - TOTAL 1.2610 K/FT

- LIVE LOAD
  - UNIFORM (0.030 KSF)(6.5 FT) = 0.1950 K/FT

- END BEAM (FRAME 1-1)
  - SPAN = 12'-0"
  - TRANSVERSE WIDTH = 3'-0"
  - FLOOR
    - DEAD LOAD
      - SLAB (3.75 FT)(5.25 IN)(1 FT/12 IN)(.150 K/FT³) = 0.2461 K/FT
      - BEAM (18.75 IN)(18 IN)(1 FT²/144 IN²)(.150 K/FT³) = 0.3516 K/FT
      - FACADE (14 FT)(0.080 KSF) = 1.1200 K/FT
      - TOTAL 1.7177 K/FT
    - LIVE LOAD
      - UNIFORM (0.080 KSF)(3.75 FT) = 0.3000 K/FT
  - ROOF
    - DEAD LOAD
      - SLAB (3 FT)(5.25 IN)(1 FT/12 IN)(0.150 K/FT³) = 0.1967 K/FT
      - BEAM (18.75 IN)(12 IN)(1 FT²/144 IN²)(.150 K/FT³) = 0.2344 K/FT
      - FACADE (1.5 FT)(0.080 KSF) = [illegible] K/FT
      - TOTAL [illegible] K/FT
    - LIVE LOAD
      - UNIFORM (0.030 KSF)(3.75 FT) = 0.1950 K/FT

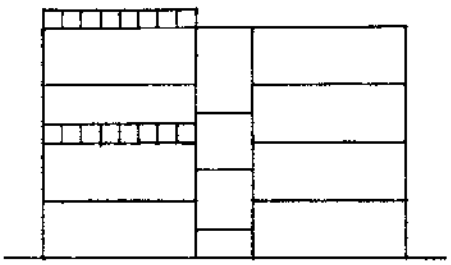
- INTERIOR BEAM (FRAME 2-2 - STAIRS)
  - SPAN = 12'-0"
  - TRANSVERSE WIDTH = 15'-0"
  - FLOOR
    - DEAD LOAD
      - SLAB (3 FT)(5.25 IN)(1 FT/12 IN)(0.150 K/FT³) = 0.1969 K/FT
      - BEAM (18.75 IN)(18 IN)(1 FT²/144 IN²)(0.150 K/FT³) = 0.3516 K/FT
      - STAIRS (12 FT)(5.25 IN)(1 FT/12 IN)(0.150 K/FT³) = 0.7875 K/FT
      - TOTAL 1.3360 K/FT
    - LIVE LOAD
      - UNIFORM (0.080 KSF)(15 FT) = 1.200 K/FT

- LIVE LOAD
  - UNIFORM (0.080 KSF)(3FT) = 0.2400 K/FT
  - CONC. LOAD ON SPAN (16FT)(0.9600 K/FT) = 15.3600 K

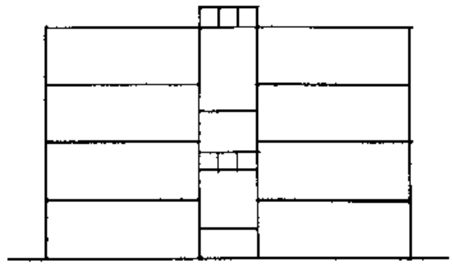
- ROOF
  - DEAD LOAD
    - SLAB (6FT)(5.25IN)(1FT/12IN)(.150 K/FT³) = 0.3938 K/F
    - BEAM (18.75IN)(12IN)(1FT²/144IN²)(.150 K/FT³) = 0.2344 K/F
    - TOTAL 0.6282 K/F
    - CONC LOAD ON SPAN (16FT)(1.0219 K/FT)+(6FT)(.1800 K/FT) = 20.1196 K
  - LIVE LOAD
    - UNIFORM (0.030 KSF)(6FT) = 0.1800 K/FT
    - CONC LOAD ON SPAN (16FT)(0.3600 K/F)+(6FT)(.1800 K/F) = 6.8400 K

LIVE LOAD PATTERNS

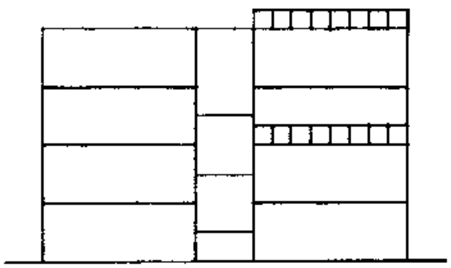
FRAME 1-1



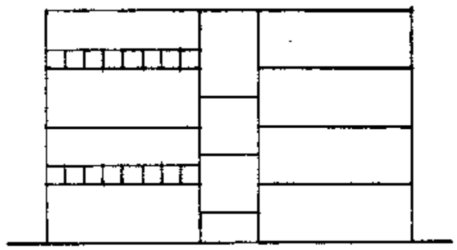
LOAD CONDITION #2



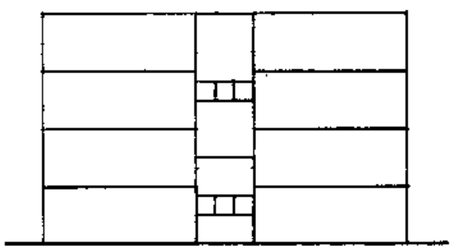
LOAD CONDITION #3



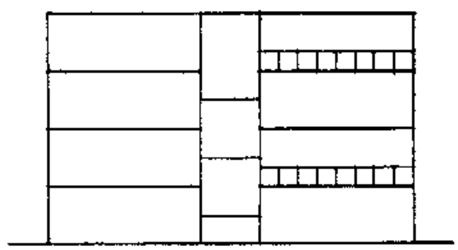
LOAD CONDITION #4



LOAD CONDITION # 5

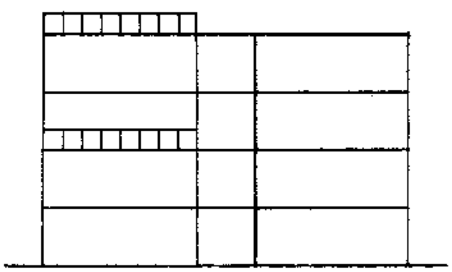


LOAD CONDITION #6

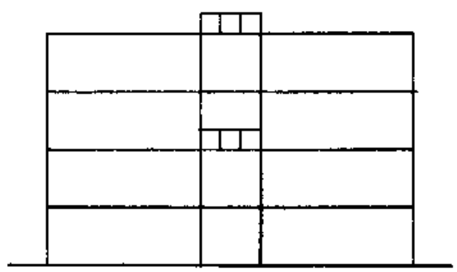


LOAD CONDITION #7

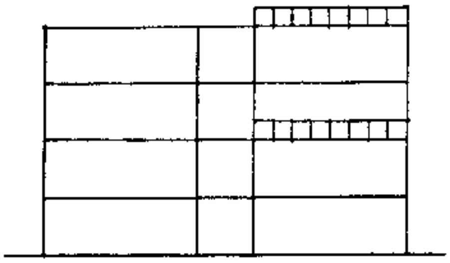
FRAMES 2-2, 3-3, 4-4, 5-5, 6-6, 7-7, 8-8, 9-9



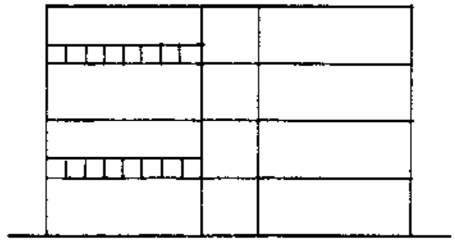
LOAD CONDITION # 2



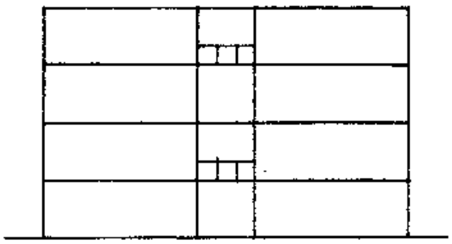
LOAD CONDITION #3



LOAD CONDITION #4



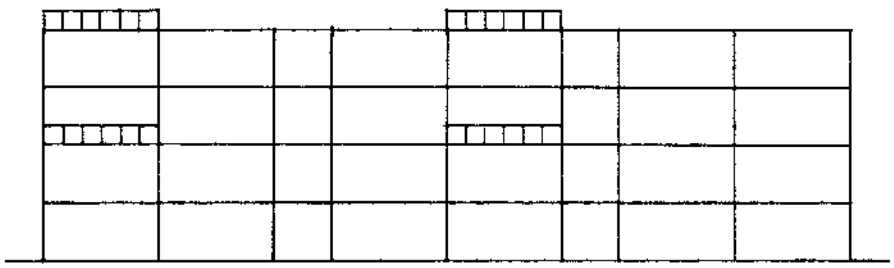
LOAD CONDITION #5



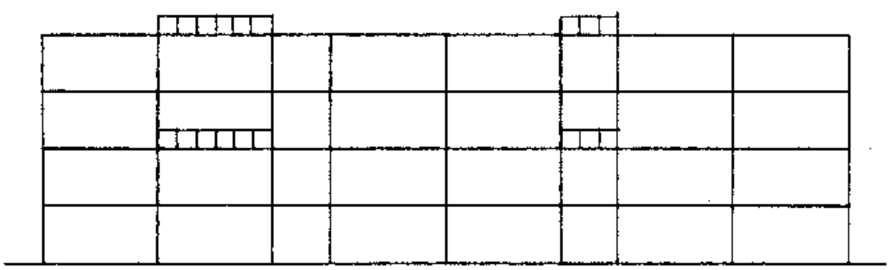
LOAD CONDITION #6

LOAD CONDITION #7

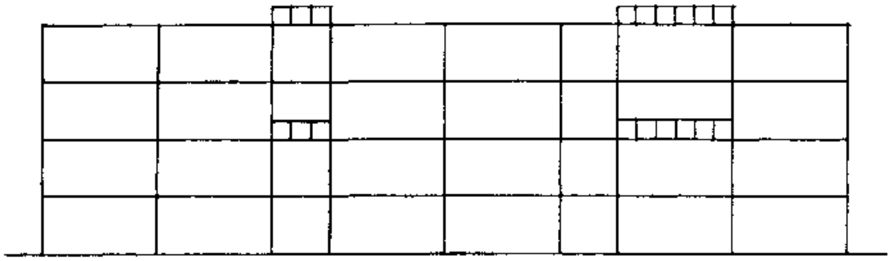
FRAMES A-A, B-B, C-C, D-D



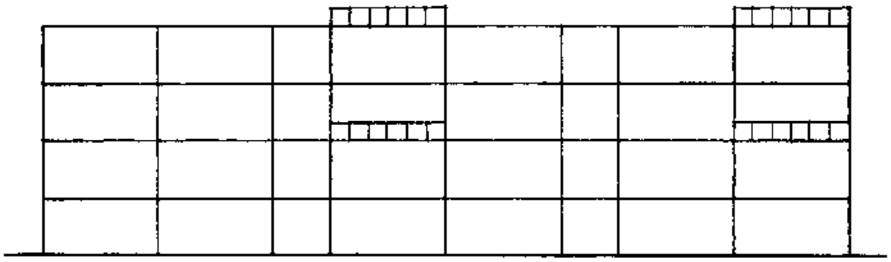
LOAD CONDITION #2



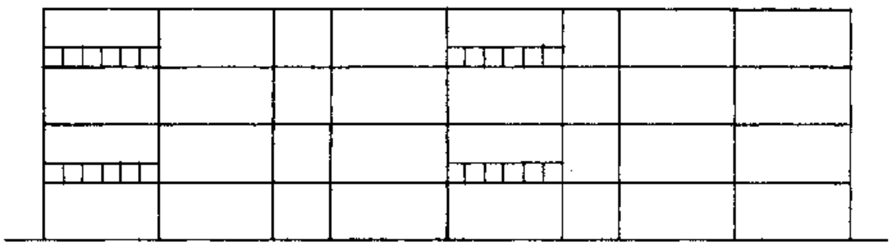
LOAD CONDITION #3



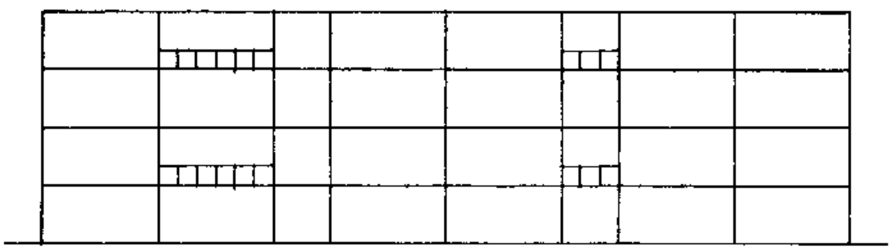
LOAD CONDITION #4



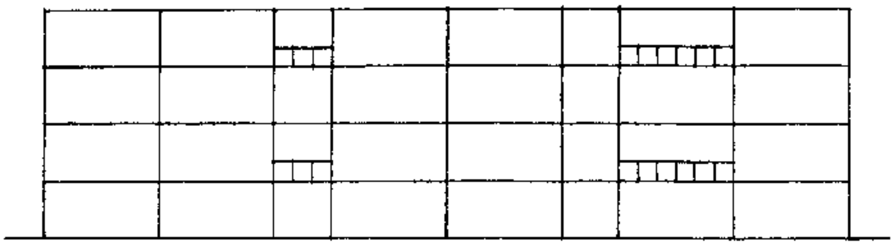
LOAD CONDITION #5



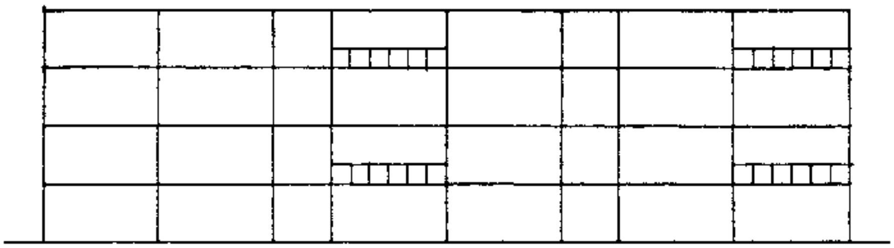
LOAD CONDITION #6



LOAD CONDITION #7



LOAD CONDITION #8



LOAD CONDITION #9

ONE-WAY SLAB DESIGN

DESIGN (FLOOR)

PROPERTIES

$f'_c = 4000$ PSI
$f_Y = 60000$ PSI
LIVE LOAD = 80 PSF

MINIMUM THICKNESS

TABLE 9.5(a) OF ACI 318-83 – MINIMUM THICKNESS OF NON-PRESTRESSED ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED

TABLE 9.5(a)
MINIMUM DEPTH, h

MEMBER	ONE-END CONTINUOUS	BOTH ENDS CONTINUOUS
SOLID ONE-WAY SLABS	$\frac{L}{24}$	$\frac{L}{28}$

$W_c = 145$ PCF
L = SPAN IN INCHES

$$\text{MIN } h = \frac{L}{24} = \frac{(12\text{ FT})(12\text{ IN./FT})}{24} = 6.000\text{ IN.} \qquad (\text{FOR S1})$$
$$\text{MIN } h = \frac{L}{28} = \frac{(12\text{ FT})(12\text{ IN./FT.})}{28} = 5.143\text{ IN.} \qquad (\text{FOR S2 AND S3})$$

ASSUME A 5¼" THICK SLAB IN SPANS S1, S2, & S3 BUT CHECK THE DEFLECTION IN THE END SPAN.

BENDING MOMENT REQUIREMENT

WIDTH OF SUPPORTING BEAMS = 18 IN.

$$W_D = (5.25\text{ IN})(1\text{ FT.}/12\text{ IN.})(0.150\text{ KCF})(1.4) = 0.092\text{ K/FT./FT. OF WIDTH}$$
$$W_L = (0.080\text{ KSF})(1.7) = 0.136\text{ K/FT./FT. OF WIDTH}$$

CLEAR SPAN = (12 FT.) – (18/12 FT) = 10.500 FT.

$$M_u = \frac{w\,l_n^2}{10} = \frac{(0.092+0.136)(10.500)^2}{10} = 2.514\text{ FT.-K / FT. OF WIDTH}$$

FOR REASONABLE DEFLECTION CONTROL CHOOSE A REINFORCEMENT PERCENTAGE $\rho = 0.375\rho_{max}$ (ONE-HALF OF THE MAXIMUM PERMITTED BY THE ACI CODE.)

$$0.375\rho_{max} = (0.375)(0.0214) = 0.0080$$

$$m = \frac{f_y}{0.85 f'_c} = \frac{60000}{(.85)(4000)} = 17.647$$

$$R_u = \rho f_y (1 - \tfrac{1}{2}\rho m)$$
$$= (0.0080)(60000)[1 - (0.5)(0.0080)(17.647)] = 446 \text{ PSI}$$

$$\text{REQ'D } d = \sqrt{\frac{M_u}{\phi R_u b}} = \sqrt{\frac{(2.514)(12000)}{(0.90)(446)(12)}} = 2.503 \text{ IN.}$$

ASSUMING #4 BARS, REQ'D $h = 2.503 + 0.250 + 0.750 = 3.503$ IN.

USE $h = 5.250$ IN. PROVIDED $d = 5.250 - 0.250 - 0.750 = 4.250$ IN.

SHEAR REQUIREMENT

$$\text{MAX. } V_u = \frac{1.15 w_u L_n}{2} = \frac{(1.15)(0.092 + 0.136)(10.500)}{2} = 1.377 \text{ K/FT. OF WIDTH}$$

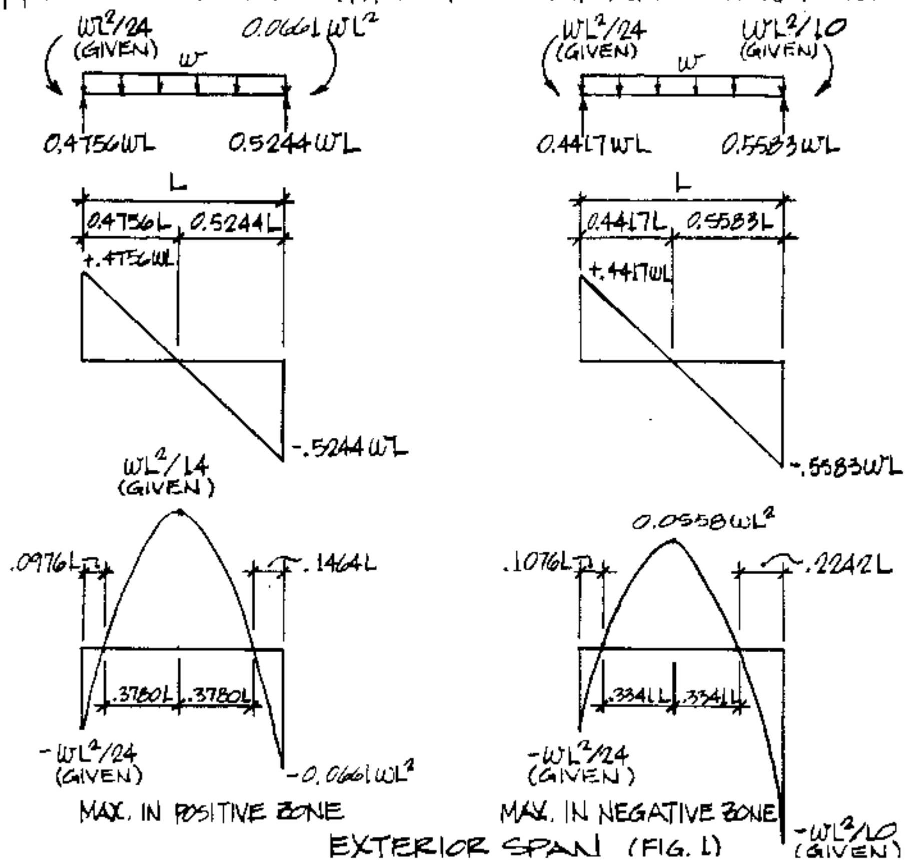
DESIGN SHEAR STRENGTH FOR A MEMBER WITHOUT SHEAR REINF. IS

$$\phi V_c = \phi[2\sqrt{f'_c}\, bd] = (.85)[(2)(\sqrt{4000})(12)(4.25)](1/1000)$$
$$= 5.483 \text{ K/FT} > 1.377 \text{ K/FT} \quad \therefore \text{ NO SHEAR REINF. IS REQ'D.}$$

AREA REQUIREMENTS

SEE TABLE 1, NEXT PAGE

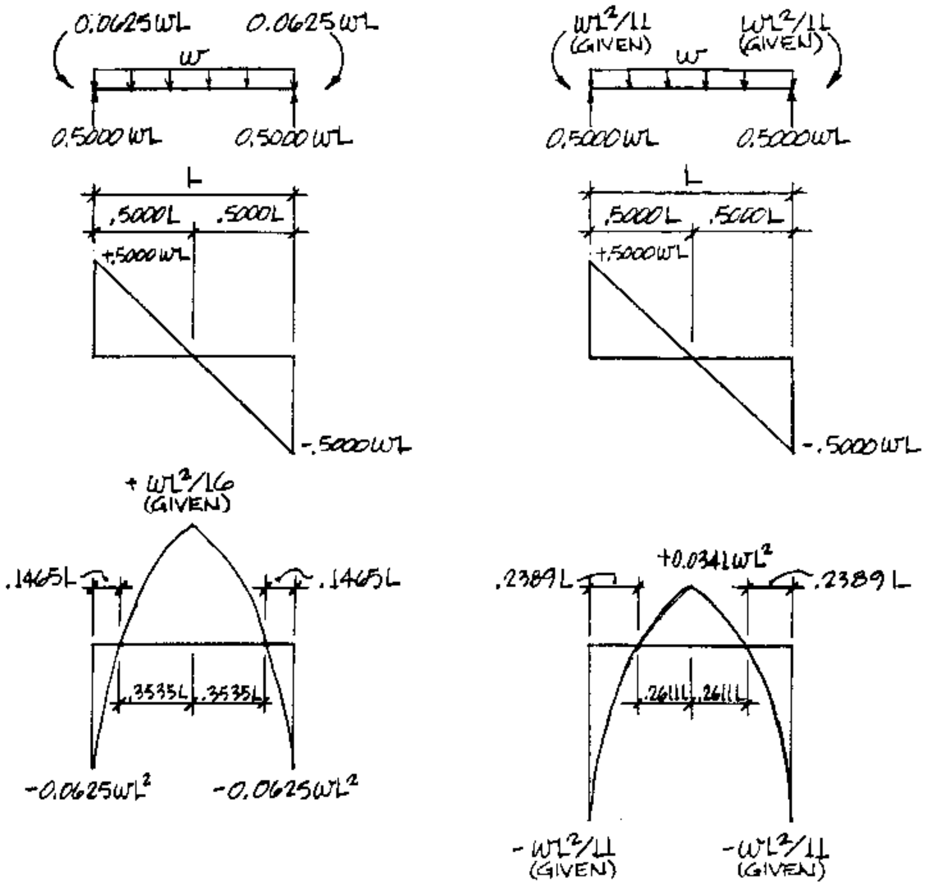
DEVELOPMENT LENGTH REQ'MTS. AT INFLECTION POINTS



EXTERIOR SPAN (FIG. 1)

TABLE 1 - FLOOR SLABS

LINE NUMBER	S1 SUPPORT	S1 MIDDLE	S1 SUPPORT	S2 SUPPORT	S2 MIDDLE	S2 SUPPORT	S3 SUPPORT	S3 MIDDLE	S3 SUPPORT
1.) ACI MOMENT COEFFICIENT	-1/24	+1/14	-1/10	-1/11	+1/16	-1/11	-1/11	+1/16	-1/11
2.) M_u (FT.-KIPS) = LINE (1) x 0.228 x $(10.500)^2$	-1.047	+1.796	-2.514	-2.285	+1.571	-2.285	-2.285	+1.571	-2.285
3.) REQ'D $R_u = \dfrac{\text{LINE (2)} \times 12000}{0.9 \times 12 \times (4.25)^2}$ = LINE (2) x 61.515	64.406	110.481	154.648	140.561	96.640	140.561	140.561	96.640	140.561
4.) REQ'D $\rho \approx \dfrac{\text{LINE (3)} \times 0.0080}{446} = \dfrac{\text{LINE (3)}}{55750.0}$	0.0012 (0.0018 MIN.)	0.0020	0.0028	0.0025	0.0017 (0.0018 MIN.)	0.0025	0.0025	0.0017 (0.0018 MIN.)	0.0025
5.) REQ'D A_s = LINE (4) x 12 x 4.25 (SQ.IN./FT.) = LINE (4) x 51.000	0.0918	0.1020	0.1428	0.1275	0.0918	0.1275	0.1275	0.0918	0.1275
6.) PROVIDED A_s	#4@18" (0.1333)	#4@18" (0.1333)	#4@16" (0.1500)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)



MAX. IN POSITIVE ZONE MAX. IN NEGATIVE ZONE

INTERIOR SPAN (FIG. 2)

FROM THE ACI SHEAR & MOMENT DIAGRAMS IN FIG.S 1 & 2, THE SHEARS AT INFLECTION POINTS ON THE TYPICAL 1-FT WIDTH OF SLAB ARE

$$V_1 = V_2 = 0.3780\,wL_n = (0.3780)(0.228)(10.500) = 0.905 \text{ KIPS}$$

$$V_3 = V_4 = V_5 = V_6 = 0.3535\,wL_n = (0.3780)(0.228)(10.500) = 0.846 \text{ KIPS}$$

FOR #4 BARS THAT EXTEND PAST THE INFLECTION POINTS INTO THE SUPPORTS, THE REQ'D. DEVELOPMENT LENGTH :

$$L_d(\#4) = 12 \text{ IN. (MIN.)}$$

THE REQM'T. OF ACI - 12.12.3 AT INFLECTION POINTS (SUCH AS PT. 1)

$$\frac{M_n}{V_u} + L_a \geq L_d$$

FOR #4 @ 16" O.C.

$$C = 0.85 f'_c b a = (.85)(4)(12)(a) = 40.800\,a$$

$$T = A_s f_Y = (.150)(60) = 9.000 \text{ KIPS}$$

$$a = \frac{9.000}{40.800} = 0.221 \text{ IN.}$$

$$M_n = C \text{ or } T\left(d - \frac{a}{2}\right) = 9.000\left(4.250 - \frac{0.221}{2}\right)\left(\frac{1}{12}\right) = 3.105 \text{ FT.-K/FT.}$$

V_u = 0.905 KIPS (COMPUTED PREVIOUSLY)

L_a = $12 d_b$ OR EFFECTIVE DEPTH = 6 IN.

$$\frac{M_n}{V_u} + L_a = \frac{(3.105)(12)}{(0.905)} + 6 = 41.171 + 6 = 47.171 \text{ IN} > L_d = 12 \text{ IN.} \quad \therefore \text{ ok}$$

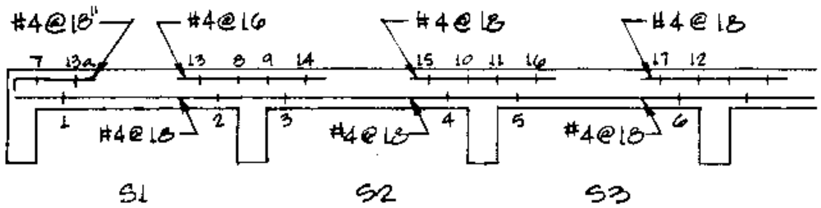
THIS CALCULATION APPLIES SIMARLY TO OTHER INFLECTION POINTS (2 THRU 6).

FOR NEGATIVE MOMENT REINFORCEMENT THE DISTANCE FROM THE FACE OF THE SUPPORT TO THE CUTOFF LOCATION MUST BE (1) GREATER THAN L_d, AND (2) ADEQUATE TO SATISFY ACI-12.13.3.

ACI-12.13.3

1) EFFECTIVE DEPTH = 5.250 IN.
2) $12 d_b$ = (12)(.5) = 6.000 IN.
3) 1/16 CLR. SPAN = (1/16)(10.5)(12) = 7.875 IN.

$\therefore L_d$ = 12 IN. (DIST. PAST POINTS 13a, 13-17)

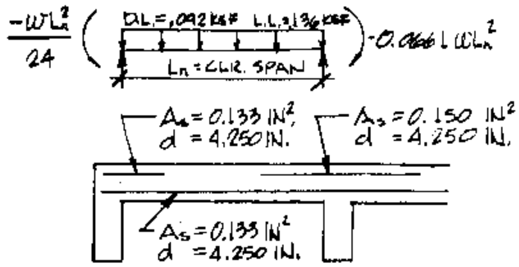


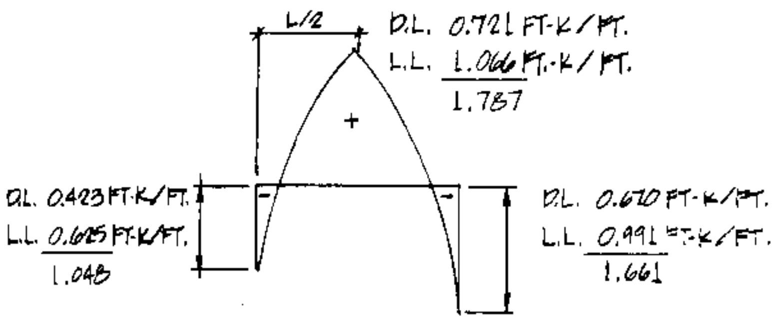
INFLECTION POINT LOCATIONS (FROM FACE OF SUPPORT)

SPAN S1	COEFF x Ln		+9"	COEFF x L
PT. 1	(0.0976)(10.5') =	1.0248'	1.7748'	0.1479 L
2	(0.1464)(10.5') =	1.5374'	2.2872'	0.1906 L
13a	(0.1076)(10.5') =	1.1298'	1.8789'	0.1567 L
13	(0.2242)(10.5') =	2.3541'	3.1041'	0.2587 L

SPANS S2,S3	COEFF. x Ln		+9"	COEFF x L
PT. 3,5	(0.1465)(10.5) =	1.5383'	2.2883'	0.1907 L
4,6	(0.1465)(10.5) =	1.5383'	2.2883'	0.1907 L
14,16	(0.2389)(10.5) =	2.5085'	3.2585'	0.2715 L
15,17	(0.2389)(10.5) =	2.5085'	3.2585'	0.2715 L

II CHECK DEFLECTION IN THE END SPAN - S1
FOR CASE 1 (REF. FIG. 1)





F'_c = 4000 PSI
F_Y = 60000 PSI
$E_c = 57000\sqrt{F'_c} = 57000\sqrt{4000}$ = 3604997 PSI $\therefore n = 8$
LIVE LOAD SUSTAINED = 33.333 %

DETERMINE I_g & I_{CR}, USING A 1-FT WIDTH OF SECTION

$$I_g = \frac{bh^3}{12} = \frac{(12)(5.25)^3}{12} = 144.703 \text{ IN.}^4$$

FOR LEFT NEGATIVE REGION, $d = 4.250$ IN., $A_s = 0.133$ IN.^{2}, $n = 8$

$$\frac{12x^2}{2} = (8)(.133)(4.250 - x)$$
$$6x^2 + 1.067x - 4.533$$
$$x = 0.785 \text{ IN.}$$

$$I_{CR} = (1/3)(12)(0.785)^3 + (8)(.133)(4.250 - 0.785)^2 = 14.742 \text{ IN.}^4$$

FOR RIGHT NEGATIVE REGION, $d = 4.250$ IN., $A_s = 0.160$ IN.^{2}, $n = 8$

$$\frac{12x^2}{2} = (8)(.150)(4.250 - x)$$
$$6x^2 + 1.200x - 5.100$$
$$x = 0.827 \text{ IN.}$$

$$I_{CR} = (1/3)(12)(0.827)^3 + (8)(.150)(4.250 - 0.827)^2 = 16.323 \text{ IN.}^4$$

FOR POSITIVE MOMENT REGION, $d = 4.250$ IN., $A_s = 0.133$ IN.^{2}, $n = 8$

$$\frac{12x^2}{2} = (8)(.133)(4.250 - x)$$

$$6x^2 + 1.066x - 4.533$$

$$x = 0.785 \text{ IN.}$$

$$I_{cr} = (1/3)(12)(0.785)^3 + (8)(.133)(4.250 - 0.785)^2 = 14.740 \text{ IN.}^4$$

DETERMINE EFFECTIVE MOMENT OF INERTIA I_e.

$$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{4000} = 474 \text{ PSI}$$

$$M_{cr} = \frac{f_r I_g}{y_t} = \frac{(.474)(144.703)}{(5.250/2)(12)} = 2.177 \text{ FT-K.}$$

$$I_e = \left(\frac{M_{cr}}{M_{max}}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_{max}}\right)^3\right] I_{cr}$$

IN LEFT NEGATIVE REGION, FOR DEAD LOAD

$$\frac{M_{cr}}{M_D} = \frac{M_{cr}}{M_{max}} = \frac{2.177}{0.423} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

& FOR DEAD LOAD PLUS LIVE LOAD

$$\frac{M_{cr}}{M_{D+L}} = \frac{M_{cr}}{M_{max}} = \frac{2.177}{1.048} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

IN RIGHT NEGATIVE REGION, FOR DEAD LOAD

$$\frac{M_{cr}}{M_D} = \frac{M_{cr}}{M_{max}} = \frac{2.177}{0.670} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

& FOR DEAD LOAD PLUS LIVE LOAD

$$\frac{M_{cr}}{M_{D+L}} = \frac{M_{cr}}{M_{max}} = \frac{2.177}{1.661} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

IN POSITIVE MOMENT REGION, FOR DEAD LOAD

$$\frac{M_{cr}}{M_D} = \frac{M_{cr}}{M_{max}} = \frac{2.177}{0.721} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

& FOR DEAD LOAD PLUS LIVE LOAD

$$\frac{M_{cr}}{M_{D+L}} = \frac{M_{cr}}{M_{max}} = \frac{2.177}{1.787} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

CONSIDER THE EFFECT OF SUSTAINED LIVE LOAD THAT CONTRIBUTES TO THE CREEP AND SHRINKAGE DEFLECTION. THE IMMEDIATE DEFLECTION DUE TO ALL SUSTAINED LOADS IS REQUIRED AS THE BASE VALUE ON WHICH TO APPLY THE TIME-DEPENDENT MULTIPLIER.

FOR LEFT NEGATIVE-MOMENT REGION

SUSTAINED LOAD MOMENT = 0.423 + (.333)(0.625) = 0.631 FT-K/FT.

$$\frac{M_{cr}}{M_{max}} = \frac{2.177}{0.631} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

FOR RIGHT NEGATIVE MOMENT REGION

SUSTAINED LOAD MOMENT = 0.670 + (.333)(.991) = 1.000 FT-K/FT.

$$\frac{M_{CR}}{M_{MAX}} = \frac{2.177}{1.000} > 1 \qquad \therefore\ I_e = I_g = 144.703\ \text{IN.}^4$$

FOR POSITIVE MOMENT REGION

SUSTAINED LOAD MOMENT = 0.721 + (.333)(1.066) = 1.076 FT-K/FT.

$$\frac{M_{CR}}{M_{MAX}} = \frac{2.177}{1.076} > 1 \qquad \therefore\ I_e = I_g = 144.703\ \text{IN.}^4$$

IMMEDIATE LIVE-LOAD DEFLECTION

$\Delta_L = \Delta_{D+L} - \Delta_D$

AVG I_e (DEAD LOAD) = 144.703 IN.^{4}

AVG. I_e (DEAD + LIVE LOAD) = 144.703 IN.^{4}

$$\text{MIDSPAN DEFLECTION} = \Delta_M = \frac{5L^2}{48EI}\left[M_s - \frac{1}{10}(M_a + M_b)\right]$$

$$\Delta_{M(D+L)} = \frac{(5)(10.5)^2(144)}{(48)(3.605\times10^3)(144.703)}\left[1.787 - \frac{1}{10}(1.048 + 1.661)\right](12) = 0.058\ \text{IN}$$

$$\Delta_{M(D)} = \frac{(5)(10.5)^2(144)}{(48)(3.605\times10^3)(144.703)}\left[0.721 - \frac{1}{10}(0.423 + 0.670)\right](12) = 0.023\ \text{IN}$$

$$\Delta_{M(L)} = 0.058 - 0.023 = 0.035\ \text{IN.}$$

CREEP AND SHRINKAGE DEFLECTION.

THE IMMEDIATE DEFLECTION DUE TO SUSTAINED LOADS

AVG. I_e = 144.703 IN^4

$$\Delta_{M(D+SUST.\,L)} = \frac{(5)(10.5)^2(144)}{(48)(3.605\times10^3)(144.703)}\left[1.076 - \frac{1}{10}(0.631 + 1.000)\right](12) = 0.035\ \text{IN.}$$

$$\Delta_{M\,(CREEP\ \&\ SHRINK)} = k_r T\, \Delta_{M(D+SUST.\,L)}$$

$$k_r T = 2 - 1.2\left(\frac{A_s'}{A_s}\right) \geq 0.6 \qquad \text{(ACI 9.5.2.5)}$$

SINCE $A_s' = 0$ $\quad k_r T = 2$

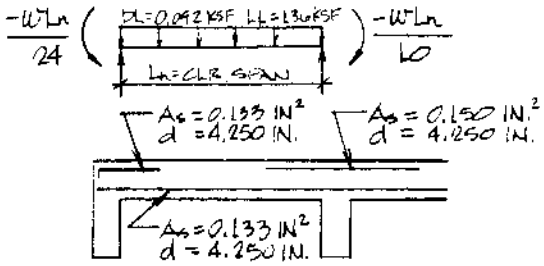
$$\therefore\ \Delta_{M\,(CREEP\ \&\ SHRINK)} = (2.0)(0.035\ \text{IN.}) = 0.070\ \text{IN.}$$

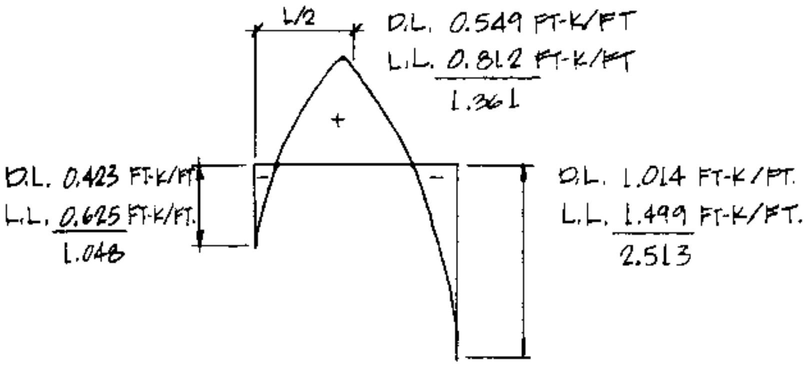
CHECK DEFLECTION TO SEE IF IT IS LESS THAN THE LIMIT OF L/480 AS PRESCRIBED BY ACI TABLE 9.5(b)

$$\Delta_{M(L)} + \Delta_{(CREEP\ \&\ SHRINK)} = 0.035 + 0.070 = 0.105\ \text{IN.}$$

$$\text{ALLOWABLE } \Delta = \frac{L}{480} = \frac{(10.5)(12)}{480} = 0.263\ \text{IN.} \qquad \therefore\ \Delta \text{ IS OK}$$

CHECK DEFLECTION IN THE END SPAN - S1 FOR CASE 2 (REF. FIG. 2)





$F'_c = 4000$ PSI
$F_Y = 60000$ PSI
$E_c = 3.605 \times 10^3$ PSI
LIVE LOAD SUSTAINED = 33.333%

DETERMINE I_g & I_{cr}, USING A 1-FT. WIDTH OF SECTION

$I_g = 144.703$ IN.⁴ (AS BEFORE)

FOR LEFT NEGATIVE REGION, $d = 4.250$ IN., $A_s = 0.133$ IN.², $n = 8$
$I_{cr} = 14.740$ IN.⁴ (AS BEFORE)

FOR RIGHT NEGATIVE REGION, $d = 4.250$ IN., $A_s = 0.150$ IN.², $n = 8$
$I_{cr} = 16.323$ IN.⁴ (AS BEFORE)

FOR POSITIVE MOMENT REGION, $d = 4.250$ IN., $A_s = 0.133$ IN.², $n = 8$
$I_{cr} = 14.740$ IN.⁴ (AS BEFORE)

DETERMINE EFFECTIVE MOMENT OF INERTIA I_e

$f_r = 474$ PSI (AS BEFORE)
$M_{cr} = 2.177$ FT-K (AS BEFORE)

$$I_e = \left(\frac{M_{CR}}{M_{MAX}}\right)^3 I_g + \left[1 - \left(\frac{M_{CR}}{M_{MAX}}\right)^3\right] I_{CR}$$

IN LEFT NEGATIVE REGION, FOR DEAD LOAD

$$\frac{M_{CR}}{M_D} = \frac{M_{CR}}{M_{MAX}} = \frac{2.177}{0.423} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

& FOR DEAD LOAD PLUS LIVE LOAD

$$\frac{M_{CR}}{M_{D+L}} = \frac{M_{CR}}{M_{MAX}} = \frac{2.177}{1.048} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

IN RIGHT NEGATIVE REGION, FOR DEAD LOAD

$$\frac{M_{CR}}{M_D} = \frac{M_{CR}}{M_{MAX}} = \frac{2.177}{1.014} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

& FOR DEAD LOAD PLUS LIVE LOAD

$$\frac{M_{CR}}{M_{D+L}} = \frac{M_{CR}}{M_{MAX}} = \frac{2.177}{2.513} = 0.866 \quad \left(\frac{M_{CR}}{M_{MAX}}\right)^3 = 0.650$$

$$I_e = (0.650)(144.703) + [1 - 0.650](16.323) = 99.770 \text{ IN.}^4$$

IN POSITIVE MOMENT REGION, FOR DEAD LOAD

$$\frac{M_{CR}}{M_D} = \frac{M_{CR}}{M_{MAX}} = \frac{2.177}{0.549} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

& FOR DEAD LOAD PLUS LIVE LOAD

$$\frac{M_{CR}}{M_{D+L}} = \frac{M_{CR}}{M_{MAX}} = \frac{2.177}{1.361} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

CONSIDER THE EFFECT OF SUSTAINED LIVE LOAD THAT CONTRIBUTES TO THE CREEP & SHRINKAGE DEFLECTION. THE IMMEDIATE DEFLECTION DUE TO ALL SUSTAINED LOADS IS REQUIRED AS THE BASE VALUE ON WHICH TO APPLY THE TIME-DEPENDENT MULTIPLIER.

FOR LEFT NEGATIVE MOMENT REGION

SUSTAINED LOAD MOMENT = $0.423 + (.333)(0.625) = 0.631$ FT-K/FT.

$$\frac{M_{CR}}{M_{MAX}} = \frac{2.177}{0.631} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

FOR RIGHT NEGATIVE MOMENT REGION

SUSTAINED LOAD MOMENT = $1.014 + (.333)(1.499) = 1.514$ FT-K/FT.

$$\frac{M_{CR}}{M_{MAX}} = \frac{2.177}{1.514} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

FOR POSITIVE MOMENT REGION

SUSTAINED LOAD MOMENT = $0.549 + (.333)(.812) = 0.820$ FT-K/FT.

$$\frac{M_{CR}}{M_{MAX}} = \frac{2.177}{1.514} > 1 \quad \therefore I_e = I_g = 144.703 \text{ IN.}^4$$

IMMEDIATE LIVE-LOAD DEFLECTION

$\Delta_L = \Delta_{D+L} - \Delta_D$

$$\text{AVG } I_e \text{ (DEAD LOAD)} = \frac{(3)(144{,}703)}{(3)} = 144{,}703 \text{ IN.}^4$$

$$\text{AVG } I_e \text{ (DEAD+LIVE)} = \frac{(144{,}703)(2)+(99{,}770)}{3} = 129{,}725 \text{ IN.}^4$$

$$\text{MIDSPAN DEFLECTION} = \Delta_m = \frac{5L^2}{48EI}\left[M_s - \frac{1}{10}(M_a+M_b)\right]$$

$$\Delta_{M(D+L)} = \frac{(5)(10.5)^2(144)}{(48)(3{,}605\times10^3)(129{,}725)}\left[1.361 - \frac{1}{10}(1.048+2.513)\right](12) = 0.043 \text{ IN.}$$

$$\Delta_{M(D)} = \frac{(5)(10.5)^2(144)}{(48)(3{,}605\times10^3)(144{,}703)}\left[0.549 - \frac{1}{10}(0.423+1.014)\right](12) = 0.015 \text{ IN.}$$

$$\Delta_{M(L)} = 0.043 - 0.015 = 0.028 \text{ IN.}$$

CREEP AND SHRINKAGE DEFLECTION

THE IMMEDIATE DEFLECTION DUE TO SUSTAINED LOADS

AVG. $I_e = 144{,}703 \text{ IN.}^4$

$$\Delta_{M(D+SUST.L)} = \frac{(5)(10.5)^2(144)}{(48)(3{,}605\times10^3)(144{,}703)}\left[0.820 - \frac{1}{10}(0.631+1.514)\right](12) = 0.023 \text{ IN}$$

$$\Delta_{M(\text{CREEP \& SHRINK})} = k_rT\,\Delta_{M(D+SUST.L)}$$

$$k_rT = 2 - 1.2\left(\frac{A'_s}{A_s}\right) \geq 0.6 \qquad \text{(ACI 9.5.2.5)}$$

SINCE $A'_s = 0$ $\quad k_rT = 2$

$$\therefore \Delta_{M(\text{CREEP \& SHRINK})} = (2.0)(0.023 \text{ IN.}) = 0.046 \text{ IN.}$$

CHECK DEFLECTION TO SEE IF IT IS LESS THAN THE LIMIT OF L/480 AS PRESCRIBED BY ACI TABLE 9.5(b)

$$\Delta_{M(L)} + \Delta_{M(\text{CREEP \& SHRINK})} = 0.028 + 0.046 = 0.074 \text{ IN.}$$

$$\text{ALLOWABLE } \Delta = \frac{L}{480} = \frac{(10.5)(12)}{480} = 0.263 \text{ IN.} \qquad \therefore \Delta \text{ IS OK}$$

DESIGN (ROOF)

PROPERTIES

$f'_c = 4000$ PSI

$f_Y = 60000$ PSI

LIVE LOAD = 30 PSF

MINIMUM THICKNESS

TABLE 9.5(a) OF ACI 318-77 - MINIMUM THICKNESS OF NON-PRESTRESSED ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED.

TABLE 9.5(a)
MINIMUM DEPTH, h

MEMBER	ONE-END CONTINUOUS	BOTH ENDS CONTINUOUS
SOLID ONE-WAY SLABS	$\frac{L}{24}$	$\frac{L}{28}$

$W_c = 145$ PCF

L = SPAN IN INCHES

$$\text{MIN. } h = \frac{L}{24} = \frac{(12\text{ FT})(12\text{ IN/FT})}{24} = 6.000 \text{ IN.} \qquad (\text{FOR S4})$$

$$\text{MIN. } h = \frac{L}{28} = \frac{(12\text{ FT})(12\text{ IN/FT})}{28} = 5.143 \text{ IN.} \qquad (\text{FOR S5 AND S6})$$

ASSUME A 5¼" THICK SLAB IN SPANS S4, S5, & S6 BUT CHECK THE DEFLECTION IN THE END SPAN.

BENDING MOMENT REQUIREMENT

WIDTH OF SUPPORTING BEAMS = 12 IN.

$W_D = (5.25\text{ IN.})(1\text{ FT}/12\text{ IN})(0.150\text{ KCF})(1.4) = 0.092$ K/FT./FT. OF WIDTH

$W_L = (0.030\text{ KSF})(1.7) = 0.051$ K/FT./FT. OF WIDTH

CLEAR SPAN = (12 FT.) − (1 FT.) = 11.000 FT.

$$M_u = \frac{W \ell_n^2}{10} = \frac{(0.092 + 0.051)(11.000)^2}{10} = 1.730 \text{ FT-K/FT. OF WIDTH}$$

FOR REASONABLE DEFLECTION CONTROL CHOOSE A REINFORCEMENT PERCENTAGE $\rho = 0.375\rho_{BAL}$. (ONE-HALF OF THE MAXIMUM PERMITTED BY THE ACI CODE.)

$$0.375\rho_{BAL} = (0.375)(0.0214) = 0.0080$$

$$m = \frac{f_Y}{0.85 f'_c} = \frac{60000}{(.85)(4000)} = 17.647$$

$R_u = \rho f_y (1 - \frac{1}{2}\rho m)$
$= (0.0080)(60000)\left[1 - (0.5)(0.0080)(17.647)\right] = 446 \text{ PSI}$

$\text{REQ'D } d = \sqrt{\dfrac{M_u}{\phi R_u b}} = \sqrt{\dfrac{(1.730)(12000)}{(0.90)(446)(12)}} = 2.076 \text{ IN.}$

ASSUMING #4 BARS, REQ'D $h = 2.076 + 0.250 + 0.750 = 3.076$ IN.

USE $h = 5.250$ IN., PROVIDED $d = 5.250 - 0.250 - 0.750 = 4.250$ IN.

SHEAR REQUIREMENT

$\text{MAX. } V_u = \dfrac{1.15 W_u L_n}{2} = \dfrac{(1.15)(0.092+0.051)(11.000)}{2} = 0.904 \text{ K/FT. OF WIDTH}$

DESIGN SHEAR STRENGTH FOR A MEMBER WITHOUT SHEAR REINF. IS

$\phi V_c = \phi\left[2\sqrt{f'_c}\, bd\right] = (0.85)\left[(2)(\sqrt{4000})(12)(4.25)\right](1/1000)$
$= 5.483 \text{ K/FT.} > 0.904 \text{ K/FT.}$ ∴ NO SHEAR REINF. IS REQ'D.

AREA REQUIREMENTS

SEE TABLE 2, NEXT PAGE

DEVELOPMENT LENGTH REQ'MTS. AT INFLECTION POINTS

REFER TO FIG. 1 & FIG. 2

FROM THE ACI SHEAR & MOMENT DIAGRAMS IN FIGS 1 & 2, THE SHEARS AT INFLECTION POINTS ON THE TYPICAL 1-FT WIDTH OF SLAB ARE

$V_1 = V_2 = 0.3780\, w_u L_n = (0.3780)(0.143)(11.000) = 0.595 \text{ KIPS}$
$V_3 = V_4 = V_5 = V_6 = 0.3535\, w_u L_n = (0.3535)(0.143)(11.000) = 0.556 \text{ KIPS}$

FOR #4 BARS THAT EXTEND PAST THE INFLECTION POINTS INTO THE SUPPORTS, THE REQ'D DEVELOPMENT LENGTH :

$L_d(\#4) = 12$ IN. (MIN.)

THE REQM'T. OF ACI-12.12.3 AT INFLECTION POINTS (SUCH AS PT. 1)

$$\frac{M_n}{V_u} + L_a \geq L_d$$

FOR #4 @ 15" O.C.

$C = 0.85 f'_c b a = (0.85)(4)(12)(a) = 40.8a$
$T = A_s f_y = (0.133)(60) = 8.000 \text{ KIPS}$

$a = \dfrac{8.000}{40.800} = 0.196 \text{ IN.}$

$M_n = C \text{ or } T\left(d - \dfrac{a}{2}\right) = 8.000\left(4.250 - \dfrac{0.196}{2}\right)\left(\dfrac{1}{12}\right) = 2.768 \text{ FT-K/FT.}$

$V_u = 0.595$ KIPS (COMPUTED PREVIOUSLY)

$L_a = 12 d_b$ OR EFFECTIVE DEPTH $= 6$ IN.

TABLE 2 - ROOF SLABS

LINE NUMBER	S4 SUPPORT	S4 MIDDLE	S4 SUPPORT	S5 SUPPORT	S5 MIDDLE	S5 SUPPORT	S6 SUPPORT	S6 MIDDLE	S6 SUPPORT
1.) ACI MOMENT COEFFICIENT	-1/24	+1/14	-1/10	-1/11	+1/16	-1/11	-1/11	+1/16	-1/11
2.) M_u (FT-KIPS) $= \text{LINE(1)} \times .143 \times (11.0)^2$	-0.721	+1.236	-1.730	-1.573	+1.081	-1.573	-1.573	+1.081	-1.573
3.) REQ'D R_u $= \dfrac{\text{LINE(2)} \times 12000}{0.9 \times 12 \times (4.25)^2} = \text{LINE (2)} \times 61.515$	44.352	76.032	106.421	96.763	66.498	96.763	96.763	66.498	96.763
4.) REQ'D ρ $\approx \dfrac{\text{LINE(3)} \times 0.0080}{446} = \dfrac{\text{LINE (3)}}{55750.0}$	0.0008 (0.0018 MIN.)	0.0014 (0.0018 MIN.)	0.0019	0.0017 (0.0018 MIN.)	0.0012 (0.0018 MIN.)	0.0017 (0.0018 MIN.)	0.0017 (0.0018 MIN.)	0.0012 (0.0018 MIN.)	0.0017 (0.0018 MIN.)
5.) REQ'D A_s (SQ. IN./FT.) $= \text{LINE (4)} \times 12 \times 4.25 = \text{LINE (4)} \times 51.00$	0.0918	0.0918	0.0961	0.0918	0.0918	0.0918	0.0918	0.0918	0.0918
6.) PROVIDE A_s	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)	#4@18" (0.1333)

$$\frac{M_n}{V_u} + L_a = \frac{(2.768)(12)}{(0.595)} + 6 = 55.825 + 6 = 61.825 \text{ IN.} > L_d = 12 \text{ IN.} \quad \text{ok}$$

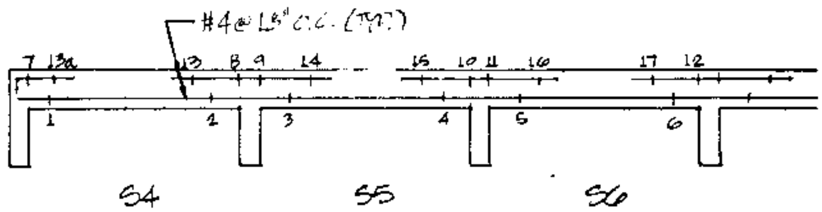
THIS CALCULATION APPLIES IDENTICALLY TO OTHER INFLECTION POINTS (2 THRU 6).

FOR NEGATIVE MOMENT REINFORCEMENT THE DISTANCE FROM THE FACE OF THE SUPPORT TO THE CUTOFF LOCATION MUST BE (1) GREATER THAN L_d, AND (2) ADEQUATE TO SATISFY ACI-12.13.3.

ACI-12.13.3

1) EFFECTIVE DEPTH = 5.250 IN.
2) $12\,d_b = (12)(.5)$ = 6.000 IN.
3) 1/16 CLR. SPAN = $(1/16)(11.000)(12) = 8.250$ IN.

$\therefore L_d = 12$ IN. (DISTANCE PAST POINTS 13a, 13-17)



INFLECTION POINT LOCATIONS (FROM FACE OF SUPPORT)

SPAN S4	COEFF. x Ln	+6"	COEFF. x L
PT. 1	(0.0976)(11.0)= 1.0736'	1.5736	0.1311 L
2	(0.1464)(11.0)= 1.6104'	2.1104	0.1759 L
13a	(0.1076)(11.0)= 1.1836'	1.6836	0.1403 L
13	(0.2242)(11.0)= 2.4662'	2.9662	0.2472 L

SPANS S5, S6	COEFF. x Ln		
PT. 3,5	(0.1465)(11.0)= 1.6115'	2.115'	0.1760 L
4,6	(0.1465)(11.0)= 1.6115'	2.115'	0.1760 L
14,16	(0.2389)(11.0)= 2.6279'	3.1279'	0.2607 L
15,17	(0.2389)(11.0)= 2.6279'	3.1279'	0.2607 L

CHECK DEFLECTION IN THE END SPAN - S4

CONSERVATIVELY

$$M_{s\,(MAX)} = \frac{wL_n^2}{8} = \frac{(.143)(11.0)^2}{8} = 2.163 \text{ FT-K} < M_{cr} = 2.177 \text{ FT-K}$$

$\therefore I_e = I_g = 144.703 \text{ IN}^4$ (FOR ALL LOAD CONDITIONS)

$$\text{MIDSPAN DEFLECTION} = \Delta_m = \frac{5L^2}{48EI}\left[M_s - \frac{1}{10}(M_a + M_b)\right]$$

LETTING $M_a = M_b = 0$

$$\Delta_m = \frac{5L^2 M_s}{48EI}$$

IMMEDIATE LIVE LOAD DEFLECTION

$$\Delta_{M(D+L)} = \frac{(5)(11.0)^2(144)(2.163)(12)}{(48)(3.605 \times 10^3)(144.703)} = 0.090 \text{ IN.}$$

$$\Delta_{M(D)} = \frac{(5)(11.0)^2(144)(1.392)(12)}{(48)(3.605 \times 10^3)(144.703)} = 0.058 \text{ IN.}$$

$$\Delta_{M(L)} = 0.090 - 0.058 = 0.032 \text{ IN.}$$

CREEP & SHRINKAGE DEFLECTION

THE IMMEDIATE DEFLECTION DUE TO SUSTAINED LOADS

$$M_{MAXIMUM\ (SUSTAINED)} = \frac{(.092 + .333 \times .051)(11.0)^2}{8} = 1.649 \text{ FT.-K}$$

$$\Delta_{M(D+SUST.\ L)} = \frac{(5)(11.0)^2(144)(1.649)(12)}{(48)(3.605 \times 10^3)(144.703)} = 0.069 \text{ IN.}$$

$$\Delta_{M(CREEP\ \&\ SHRINK)} = k_r T \Delta_{M(D+SUST.\ L)}$$

$k_r T = 2$ (AS BEFORE)

$$\therefore \Delta_{M(CREEP\ \&\ SHRINK)} = (2.0)(0.069) = 0.138 \text{ IN.}$$

CHECK DEFLECTION TO SEE IF IT IS LESS THAN THE LIMIT OF L/480 AS PRESCRIBED BY ACI TABLE 9.5 b

$$\Delta_{M(L)} + \Delta_{M(CREEP\ \&\ SHRINK)} = 0.032 + 0.138 = 0.170 \text{ IN.}$$

$$\text{ALLOWABLE } \Delta = \frac{L}{480} = \frac{(11.0)(12)}{480} = 0.275 \text{ IN.} \quad \therefore \Delta \text{ IS OK}$$

II SHRINKAGE & TEMPERATURE REINFORCEMENT

FOR SLABS S1 THRU S26

$F_Y = 60000$ PSI

USE DEFORMED BARS

ACI 7.12 $\rho \geq 0.0018$

$$\therefore A_s = \rho b d = (0.0018)(12)(4.25) = 0.0918 \text{ IN}^2/\text{FT.}$$

ACI 7.12.3

MAX. BAR SPACING = 18 IN.

USE #4 @ 18" O.C. ($A_s = 0.133$ IN.^{2}/FT.)

II NEGATIVE MOMENT REINFORCEMENT (TOP BARS)

FOR ADEQUATE RIGIDITY OF THE TOP BARS AND TO [illegible] CONSTRUCTION TRAFFIC IN MONOLITHIC CONSTRUCTION, THE 1984 CRSI HANDBOOK RECOMMENDS [illegible] 12" O.C. MINIMUM. DUE TO THIS RECOMMENDATION, #4 TOP BARS @ [illegible] O.C. WILL BE USED IN LIEU OF THE PREVIOUS CALCULATIONS.

BEAM DESIGN (LONGHAND)

BEAM # B7 (BEAM #17 FRAME 5-5)

MAXIMUM & MINIMUM MOMENTS & SHEARS @ 1/10th PTS (K & FT) FACTORED

1	2	3	4	5	6	7	8	9	10	11
-9.2	26.7	80.8	136.2	178.6	188.0	165.4	111.2	39.7	0.1	-39.7
-206.5	-89.0	-23.2	14.7	38.5	51.7	48.8	21.7	-17.4	-113.3	-265.9
50.2	39.9	29.5	19.2	9.6	4.8	12.7	23.1	33.4	43.7	54.0

DESIGN OF LONGITUDINAL REINFORCEMENT

$$d_{TOP\ BARS} = \underset{h}{24''} - \underset{COVER}{1.5''} - \underset{STIRRUP}{0.375''} - \underset{GIRDER\ BAR\ DIA.}{0.500''} - \underset{HALF\ BAR\ DIA.}{0.250''} = 21.375''$$

$$d_{BTM.\ BARS} = \underset{h}{24''} - \underset{COVER}{1.5''} - \underset{STIRRUP}{0.375} - \underset{HALF\ BAR\ DIA.}{0.500} = 21.625''$$

TEE BEAM (32' SPAN)

FLG. WIDTH	T(FLG.)	T(WEB)	TOTAL DEPTH
96"	5.250"	18.000	24"

℄ OF SUPPORT TO FACE = 10 IN. EACH END

F'_c = 4 KSI

CONC. WT. = 145 PCF

F_Y (REBARS) = 60 KSI

F_Y (STIRRUPS) = 40 KSI

SECTION A-A

$$M_u \text{ @ FACE} = 206.5 - \left(\frac{206.5-89.0}{3.2}\right)\left(\frac{10}{12}\right) = 175.901 \text{ K-FT.}$$

$$\text{REQ'D } R_n = \frac{M_u}{\phi b d^2} = \frac{(175.901)(12000)}{(0.9)(18)(21.375)^2} = 285 \text{ PSI}$$

$$m = \frac{f_Y}{0.85 f'_c} = \frac{60}{(0.85)(4)} = 17.647$$

$$\text{REQ'D } \rho = \frac{1}{m}\left(1-\sqrt{1-\frac{2mR_n}{f_Y}}\right) = \frac{1}{17.647}\left(1-\sqrt{1-\frac{(2)(17.647)(285)}{60000}}\right) = 0.004967751$$

$$\text{MIN } \rho = \frac{200}{f_Y} = \frac{200}{60000} = 0.003\bar{3}$$

$$\text{REQ'D } A_s = \rho b d = (0.004967751)(18)(21.375) = 1.911 \text{ IN}^2$$

SECTION C-C

$$M_u \text{ @ FACE} = 265.5 - \left(\frac{265.5 - 113.3}{3.2}\right)\left(\frac{10}{12}\right) = 226.160 \text{ K-FT.}$$

$$\text{REQ'D } R_n = \frac{(226.160)(12000)}{(.9)(18)(21.375)^2} = 367 \text{ PSI}$$

$$\text{REQ'D } \rho = \frac{1}{17.647}\left(1 - \sqrt{1 - \frac{(2)(17.647)(367)}{60000}}\right) = 0.006488095$$

$$\text{MIN } \rho = 0.003\overline{3}$$

$$\text{REQ'D } A_s = (0.006488095)(18)(21.375) = 2.496 \text{ IN}^2$$

SECTION B-B (MAX. POSITIVE MOMENT = 187.99 K-FT.)

$$\text{ESTIMATE MOMENT ARM} = 0.9d = (0.9)(21.625) = 19.463$$

$$\text{REQ'D } A_s = \frac{M_u}{\phi f_y (\text{ARM})} = \frac{(187.99)(12)}{(.9)(60)(19.463)} = 2.146 \text{ IN}^2$$

CHECK:

$$C = 0.85 f'_c b_E a = (0.85)(4)(96)(a) = 326.400a$$

$$T = A_s f_y = (2.146)(60) = 128.788 \text{ KIPS}$$

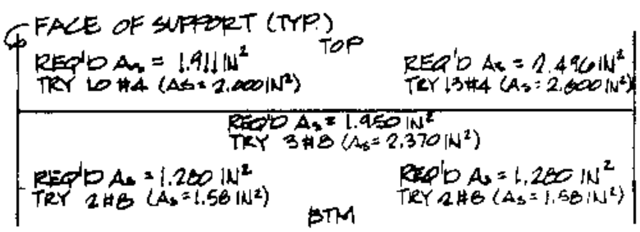
$$a = \frac{128.788}{326.400} = 0.395 \text{ IN} < T(\text{FLG})$$ DESIGN AS RECTANGULAR SECTION

$$\text{ARM} = d - a/2 = 21.625 - .395/2 = 21.428 \text{ IN.}$$

$$\text{REVISED REQ'D } A_s = \frac{(187.99)(12)}{(.9)(60)(21.428)} = 1.950 \text{ IN}^2$$

$$\text{MIN } A_s \text{ @ SUPPORT FACE} = \rho_{MIN} bd = (0.0033)(18)(21.625) = 1.297 \text{ IN}^2$$

REQ'D REINFORCEMENT



CAPACITY 3#8

$T = A_s f_y = (3)(.79)(60) = 142.200^k$

$C = 0.85 f'_c b_E a = (.85)(4)(96)(a) = 326.4a$

$C = T \therefore a = \dfrac{142.200}{326.4} = 0.436$ IN.

$M_n = C \text{ or } T\left(d - \dfrac{a}{2}\right) = 142.200\left(21.625 - \dfrac{0.436}{2}\right)\left(\dfrac{1}{12}\right) = 253.673$ K-FT.

$\phi M_n = (.9)(253.673) = 228.306$ K-FT.

CAPACITY 2#8

$T = (2)(.79)(60) = 94.800^k$

$C = (.85)(4)(96)(a) = 326.4a$

$C = T \therefore a = \dfrac{94.800}{326.4} = 0.290$ IN.

$M_n = 94.800\left(21.625 - \dfrac{0.290}{2}\right)\left(\dfrac{1}{12}\right) = 169.692$ K-FT.

$\phi M_n = (.9)(169.692) = 152.723$ K-FT.

CAPACITY 10#4

$T = (10)(.2)(60) = 120.000^k$

$C = .85 f'_c b a = (.85)(4)(18)(a) = 61.200a$

$C = T \therefore a = \dfrac{120.000}{61.200} = 1.961$ IN.

$M_n = 120.000\left(21.375 - \dfrac{1.961}{2}\right)\left(\dfrac{1}{12}\right) = 203.946$ K-FT.

$\phi M_n = (.9)(203.946) = 183.551$ K-FT.

CAPACITY 4#4

$T = (4)(.2)(60) = 48.000^k$

$C = (.85)(4)(18)(a) = 61.200a$

$C = T \therefore a = \dfrac{48.000}{61.200} = 0.784$ IN.

$M_n = 48.000\left(21.375 - \dfrac{0.784}{2}\right)\left(\dfrac{1}{12}\right) = 83.931$ K-FT

$\phi M_n = (.9)(83.931) = 75.538$ K-FT.

CAPACITY 13#4

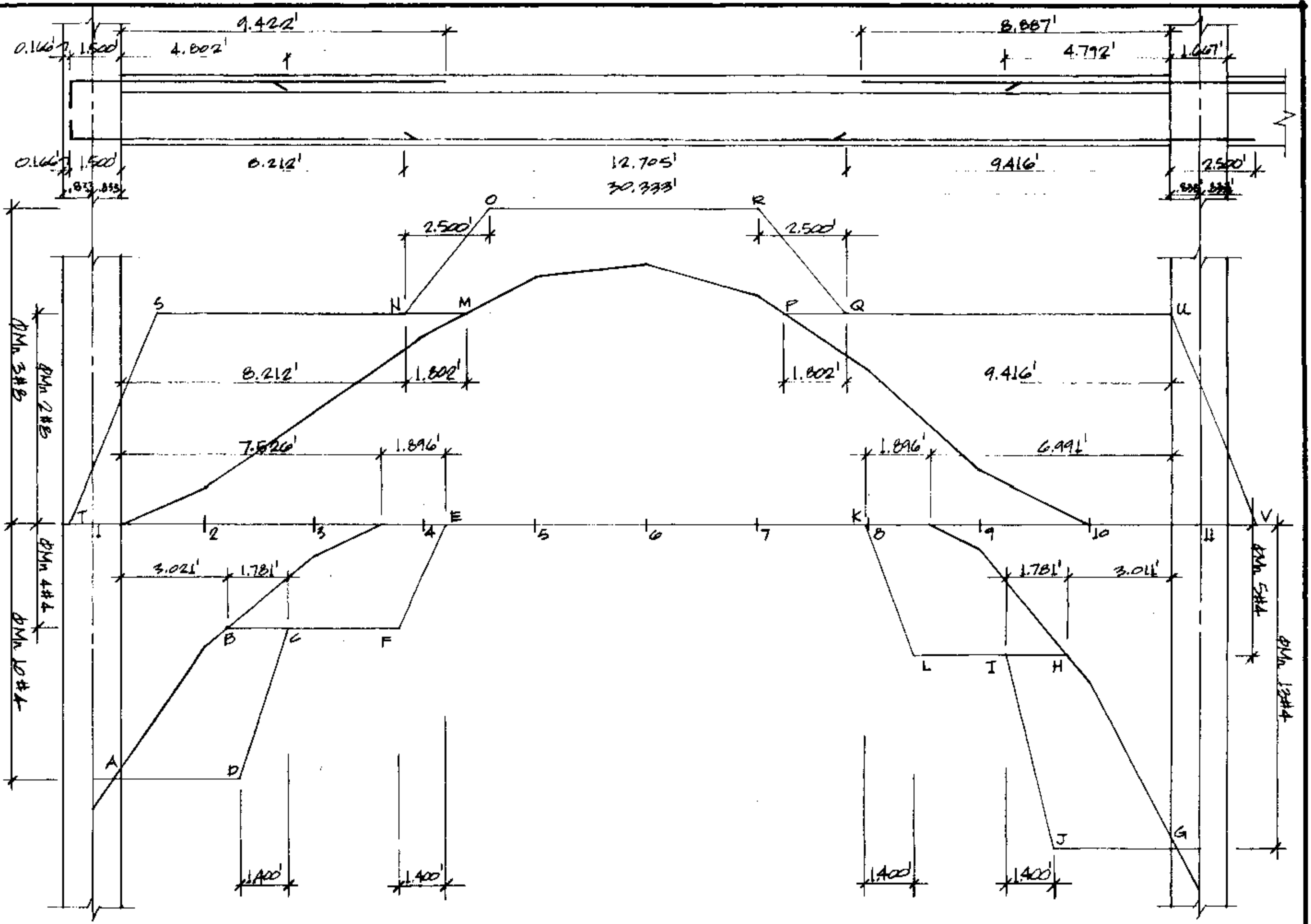
$T = (13)(.2)(60) = 156.000^k$

$C = (.85)(4)(18)(a) = 61.200a$

$C = T \therefore a = \dfrac{156.000}{61.200} = 2.549$ IN.

$M_n = 156.000\left(21.375 - \dfrac{2.549}{2}\right)\left(\dfrac{1}{12}\right) = 261.307$ K-FT

$\phi M_n = (.9)(261.307) = 235.176$ K-FT



MOMENT ENVELOPE, MOMENT CAPACITY DIAGRAM, & BAR ARRANGEMENT FOR BEAM B7 (UNITS FT & K)

CAPACITY 5#4

$T = (5)(.2)(60) = 60.000^{k}$

$C = (.85)(4)(18)(a) = 61.200\,a$

$C = T \therefore a = \dfrac{60.000}{61.200} = 0.980$ IN

$M_n = 60.000\left(21.375 - \dfrac{0.980}{2}\right)\left(\dfrac{1}{12}\right) = 104.425$ K·FT

$\phi M_n = (.9)(104.425) = 93.983$ K·FT

REFER TO THE MOMENT ENVELOPE DIAGRAM FOR CUT-OFF POINTS

DESIGN OF SHEAR (REFER TO THE SHEAR ENVELOPE DIAGRAM)

$\phi V_c = \phi(2\sqrt{f'_c})b_w d = (.85)(2)(\sqrt{4000})(18)(21.625)(1/1000) = 41.851^{k}$

$\phi V_c/2 = 20.926^{k}$

$V_1 = 50.2 - \left(\dfrac{50.2-39.9}{3.2}\right)(.833+1.802) = 41.718^{k}$ (@ d FROM LEFT SUPPORT)

$V_2 = 54.0 - \left(\dfrac{54.0-43.7}{3.2}\right)(.833+1.802) = 45.518^{k}$ (@ d FROM RIGHT SUPPORT)

MAX. REQ'D $\phi V_s = V_u - \phi V_c = 45.518 - 41.851 = 3.667^{k}$

MAX. PERMISSIBLE STIRRUP SPACING $= d/2$ WHEN $V_s < 4\sqrt{f'_c}$

LIMIT $\phi V_s = \phi(4\sqrt{f'_c}\,b_w d) = 2(\phi V_c) = 2(41.851) = 83.702^{k}$

MAX. REQ'D $\phi V_s = 3.667^{k} < 83.702^{k} =$ LIMIT $\phi V_s \therefore$ MAX. $S = d/2$

MIN. $\phi V_s = \phi 50 b_w d = (.85)(50)(18)(21.625)(1/1000) = 16.543^{k}$

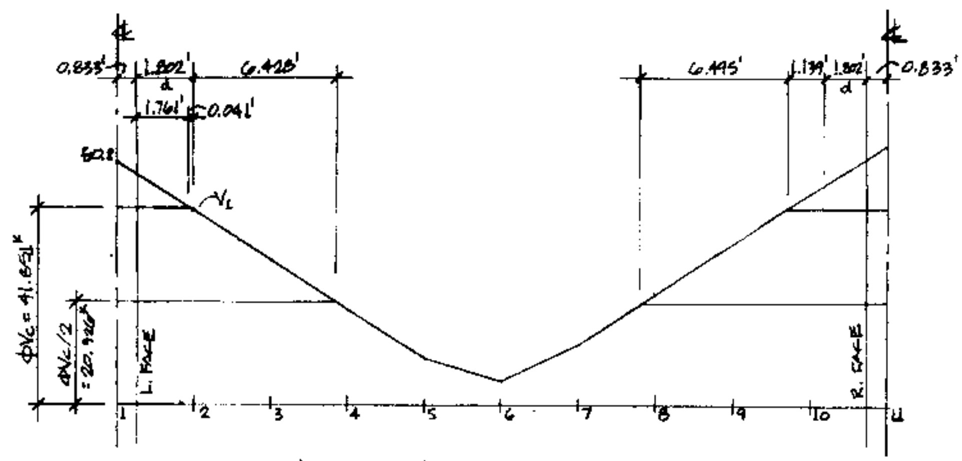
MIN. $\phi V_s = 16.543^{k} > 3.667^{k} =$ MAX. REQ'D V_s $\therefore$ USE MAX. STIRRUP SPACING FOR #3 STIRRUPS

STIRRUPS REQ'D UNTIL 8.230' FROM THE LEFT SUPPORT FACE

USE 1@5, 9@10½ (8.292')

STIRRUPS REQ'D UNTIL 9.436' FROM THE RIGHT SUPPORT FACE

USE 1@9, 10@10½ (9.500')



SHEAR ENVELOPE (UNITS FT. & K)

BEAM DESIGN MOMENTS

LOAD CONDITIONS

LC# 1	DEAD LOAD
LC# 2	LIVE LOAD COMBINATION
LC# 3	LIVE LOAD COMBINATION
LC# 4	LIVE LOAD COMBINATION
LC# 5	WIND LOAD
LC# 6	SEISMIC LOAD

UNITS

MOMENT	KIP-FEET
UNIFORM LOAD (U)	KIPS PER FOOT
CONCENTRATED LOAD (C)	KIPS

* LOADS ARE UNIFORM UNLESS DENOTED BY (C)

FRAME 1-1

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
23	1	-132.205	-142.423	1.915
	2	-4.414	3.343	0
	3	-35.603	-38.445	0.540
	4	2.423	-4.540	0
	5	16.501	-18.472	0
	6	13.634	-15.336	0
24	1	-15.513	-15.513	2.303
	2	-10.869	-10.869	1.020
	3	4.293	-1.187	0
	4	-1.187	4.293	0
	5	25.445	-24.361	0
	6	19.573	-19.254	0
25	1	-142.423	-132.205	1.915
	2	3.343	-4.414	0
	3	-4.540	2.423	0
	4	-38.445	-35.603	0.540
	5	18.493	-16.101	0
	6	15.375	-13.485	0
26	1	-141.767	-136.639	1.915
	2	-34.102	-38.137	0.540
	3	-8.934	5.368	0
	4	3.358	-4.977	0
	5	13.307	-14.461	0
	6	12.317	-13.341	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
27	1	- 3.380	- 3.380	2.3[illegible]
	2	8.069	- 1.462	0
	3	- 9.084	- 9.084	1.02[illegible]
	4	- 1.462	8.069	0
	5	23.956	- 23.674	0
	6	21.446	-20.942	0
28	1	-136.639	-141.767	1.915
	2	- 4.977	3.358	0
	3	5.308	- 8.1[illegible]	0
	4	- 38.187	- 34.902	0.5[illegible]
	5	14.379	- 12.645	0
	6	13.774	- 12.527	0
29	1	-144.243	-133.659	1.915
	2	- 40.123	- 37.141	0.540
	3	- 0.953	2.1[illegible]	0
	4	1.521	- 2.223	0
	5	9.209	- [illegible]	0
	6	10.709	- [illegible]	0
30	1	- 3.693	- 3.698	2.30[illegible]
	2	5.192	- 3.403	0
	3	- 8.4[illegible]	- 8.404	1.[illegible]
	4	- [illegible]	3.1[illegible]	0
	5	16.[illegible]	- 15.[illegible]	0
	6	17.33[illegible]	- 15.[illegible]	0
31	1	-133.659	- 144.243	1.915
	2	2.119	- 0.953	0
	3	- 37.141	- 40.123	0.5[illegible]0
	4	- 2.223	1.521	0
	5	3.651	- 3.756	0
	6	10.390	- 10.[illegible]	0
32	1	- 78.088	-101.012	1.261
	2	- 13.638	- 14.[illegible]	0.1[illegible]
	3	2.167	- 3.[illegible]	0
	4	- 2.113	1.9[illegible]	0
	5	3.30[illegible]	- 2.619	0
	6	4.337	- 3.617	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
33	1	− 40.225	− 40.225	1.031
	2	− 10.466	7.070	0
	3	− 1.547	− 1.547	0.195
	4	7.070	− 10.466	0
	5	1.096	− 2.271	0
	6	2.323	− 3.324	0
34	1	− 101.012	− 78.098	1.261
	2	1.933	− 2.193	0
	3	− 3.012	2.167	0
	4	− 14.635	− 13.633	0.195
	5	4.210	− 4.785	0
	6	4.393	− 5.331	0

FRAME 2-2

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
17	1	− 71.479	− 91.202	1.139
	2	− 8.320	6.937	0
	3	0.359	− 4.850	0
	4	− 52.363	− 79.215	0.960
	5	42.505	− 31.922	0
	6	17.613	− 13.338	0
18	1	− 47.892	− 47.892	1.336
	2	14.420	− 46.208	0
	3	− 9.330	− 9.330	1.200
	4	− 46.208	14.420	0
	5	46.419	− 46.430	0
	6	19.182	− 19.194	0
19	1	− 91.202	− 71.479	1.139
	2	− 4.850	0.359	0
	3	6.937	− 8.320	0
	4	− 79.215	− 52.363	0.960
	5	31.711	− 42.220	0
	6	13.303	− 17.540	0
20	1	− 76.643	− 90.326	1.139
	2	− 52.580	− 78.200	0.960
	3	1.100	− 6.505	0
	4	− 13.603	8.601	0
	5	32.348	− 25.515	0
	6	15.567	− 12.434	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
21	1	-43.685	-43.685	1.336
	2	-50.535	23.233	0
	3	-9.648	-9.648	1.200
	4	23.233	-50.535	0
	5	35.733	-35.727	0
	6	16.901	-16.899	0
22	1	-90.326	-76.643	1.139
	2	8.601	-13.608	0
	3	-6.808	1.100	0
	4	-78.200	-52.580	0.960
	5	25.861	-32.320	0
	6	12.405	-15.507	0
23	1	-80.296	-89.844	1.139
	2	-10.979	7.172	0
	3	1.120	-6.587	0
	4	-52.976	-76.821	0.960
	5	19.520	-16.405	0
	6	11.127	-9.205	0
24	1	-40.582	-40.582	1.336
	2	18.887	-48.639	0
	3	-9.025	-9.025	1.200
	4	-48.639	18.887	0
	5	17.695	-17.696	0
	6	10.368	-10.367	0
25	1	-89.844	-80.296	1.139
	2	-6.587	1.120	0
	3	7.172	-10.979	0
	4	-76.821	-52.976	0.960
	5	16.412	-19.532	0
	6	9.167	-11.049	0
26	1	-57.386	-79.682	1.022
	2	-17.357	-27.911	0.360
	3	1.246	-3.954	0
	4	-8.005	4.776	0
	5	7.180	-6.750	0
	6	4.756	-4.217	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
27	1	− 44.627	− 44.627	0.628
	2	− 18.448	− 18.448	0.180
	3	13.769	− 7.773	0
	4	− 7.773	13.769	0
	5	3.640	− 3.644	0
	6	2.990	− 2.992	0
28	1	− 79.682	− 57.386	1.022
	2	− 3.954	1.246	0
	3	− 22.911	− 17.357	0.360
	4	4.776	− 8.005	0
	5	6.774	− 7.231	0
	6	4.191	− 4.709	0

FRAMES 3-3, 4-4, 6-6, 7-7

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
17	1	− 71.650	− 90.028	1.139
	2	− 8.363	6.581	0
	3	0.493	− 3.064	0
	4	− 52.365	− 79.215	0.960
	5	31.625	− 23.758	0
	6	17.613	− 13.338	0
18	1	− 43.290	− 43.290	0.745
	2	14.480	− 46.208	0
	3	− 3.723	− 3.723	0.480
	4	− 46.208	14.480	0
	5	34.543	− 34.550	0
	6	19.192	− 19.194	0
19	1	− 90.028	− 71.650	1.139
	2	− 3.064	0.493	0
	3	6.581	− 8.363	0
	4	− 79.215	− 52.365	0.960
	5	23.660	− 31.421	0
	6	13.373	− 17.540	0
20	1	− 76.564	− 84.581	1.139
	2	− 52.580	− 78.200	0.960
	3	0.733	− 5.021	0
	4	− 13.145	7.721	0
	5	24.261	− 19.398	0
	6	15.507	− 12.434	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
21	1	− 38.911	− 38.911	0.745
	2	− 5[illegible].336	23.233	0
	3	− 3.832	− 3.332	0.480
	4	23.233	− 50.535	0
	5	26.773	− 26.769	0
	6	16.901	− 16.849	0
22	1	− 89.581	− 76.564	1.139
	2	7.721	− 13.145	0
	3	− 5.021	[illegible].733	0
	4	− 78.200	− 52.580	0.960
	5	19.379	− 24.228	0
	6	12.495	− 15.527	0
23	1	− 80.052	− 89.075	1.139
	2	− 10.665	6.644	0
	3	1.103	− 5.122	0
	4	− 52.976	− 76.821	0.960
	5	14.635	− 12.299	0
	6	11.127	− 9.205	0
24	1	− 36.178	− 36.178	0.745
	2	18.887	− 48.641	0
	3	− 3.661	− 3.661	0.480
	4	− 48.639	18.337	0
	5	13.292	− 13.283	0
	6	10.[illegible]	− 10.367	0
25	1	− 89.075	− 80.[illegible]	1.139
	2	− 5.122	1.[illegible]	0
	3	6.644	− [illegible]	0
	4	− 76.321	− [illegible]	0.960
	5	12.3[illegible]	− 14.[illegible]	0
	6	9.167	− 11.049	0
26	1	− 56.930	− 80.392	1.022
	2	− 17.152	− 23.115	0.360
	3	1.246	− 3.954	0
	4	− 7.655	4.127	0
	5	5.333	− 5.060	0
	6	4.760	− 4.217	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
27	1	− 44.600	− 44.600	0.625
	2	− 18.414	− 18.414	0.150
	3	13.769	− 7.713	0
	4	− 7.713	13.769	0
	5	2.779	− 2.776	0
	6	2.990	− 2.992	0
28	1	− 80.382	− 56.930	1.022
	2	− 3.954	1.246	0
	3	− 28.115	− 17.152	0.360
	4	4.127	− 7.655	0
	5	5.077	− 5.422	0
	6	4.191	− 4.709	0

FRAMES 5-5, 8-8

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
17	1	− 71.650	− 90.028	1.139
	2	− 8.363	6.581	0
	3	0.493	− 3.604	0
	4	− 52.365	− 79.215	0.960
	5	42.505	− 31.791	0
	6	17.613	− 13.323	0
18	1	− 43.210	− 43.210	0.745
	2	14.436	− 46.208	0
	3	− 8.723	− 8.724	0.480
	4	− 46.208	14.436	0
	5	46.419	− 46.420	0
	6	19.172	− 19.174	0
19	1	− 90.028	− 71.650	1.139
	2	− 3.604	0.493	0
	3	6.581	− 8.363	0
	4	− 79.215	− 52.365	0.960
	5	31.791	− 42.505	0
	6	13.323	− 17.613	0
20	1	− 16.564	− 34.581	1.139
	2	− 52.380	− 78.200	0.960
	3	0.733	− 5.021	0
	4	− 13.145	7.721	0
	5	32.348	− 25.818	0
	6	15.567	− 12.430	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
21	1	-38.911	-38.911	0.745
	2	-50.535	23.233	0
	3	-3.832	-3.832	0.480
	4	23.233	-50.535	0
	5	35.733	-35.727	0
	6	16.901	-16.899	0
22	1	-89.581	-76.564	1.139
	2	7.721	-13.145	0
	3	-5.021	6.733	0
	4	-78.200	-52.580	0.960
	5	25.861	-32.320	0
	6	12.405	-15.507	0
23	1	-80.052	-89.075	1.139
	2	-10.665	6.644	0
	3	1.103	-5.122	0
	4	-52.976	-76.821	0.960
	5	19.520	-16.405	0
	6	11.127	-9.205	0
24	1	-36.178	-36.1[illegible]	0.745
	2	18.887	-48.699	0
	3	-3.661	-3.661	0.480
	4	-48.699	18.887	0
	5	17.695	-17.696	0
	6	10.338	-10.327	0
25	1	-89.075	-80.052	1.139
	2	-5.122	1.103	0
	3	6.644	-10.[illegible]	0
	4	-76.321	-52.976	0.960
	5	16.412	-19.532	0
	6	9.167	-11.049	0
26	1	-56.930	-81.382	1.022
	2	-17.152	-23.115	0.350
	3	1.246	-3.[illegible]	0
	4	-7.655	[illegible]	0
	5	7.130	-[illegible]	0
	6	4.766	-[illegible].217	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
27	1	-44.600	-44.600	0.628
	2	-18.414	-18.414	0.180
	3	13.769	-7.773	0
	4	-7.773	13.769	0
	5	3.645	-3.074	0
	6	2.990	-2.112	0
28	1	-80.332	-56.930	1.022
	2	-3.954	1.246	0
	3	-28.115	-17.162	0.360
	4	4.127	-7.655	0
	5	6.774	-7.231	0
	6	4.191	-4.700	0

FRAME 9-9

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
17	1	-128.903	-151.097	1.915
	2	-4.237	3.055	0
	3	0.236	-1.550	0
	4	-32.305	-43.767	0.540
	5	20.344	-16.199	0
	6	16.886	-13.433	0
18	1	-62.010	-62.010	1.718
	2	8.001	-21.245	0
	3	-2.645	-2.645	0.300
	4	-21.245	8.001	0
	5	22.550	-22.548	0
	6	13.713	-13.710	0
19	1	-151.097	-128.903	1.915
	2	-1.550	0.236	0
	3	3.055	-4.237	0
	4	-43.767	-32.305	0.540
	5	16.199	-20.165	0
	6	13.440	-16.771	0
20	1	-136.753	-151.549	1.915
	2	-32.351	-43.300	0.540
	3	0.341	-2.303	0
	4	-6.759	3.293	0
	5	15.733	-13.130	0
	6	15.174	-12.344	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
21	1	- 55.467	- 55.467	1.718
	2	- 23.554	12.4[illegible]	0
	3	- 2.7[illegible]6	- [illegible]	0.540
	4	12.445	- 23.554	0
	5	17.713	[illegible]	0
	6	16.347	- [illegible]	0
22	1	- 15[illegible]	- 156.[illegible]	[illegible]
	2	[illegible]	- [illegible]	0
	3	- [illegible]	[illegible]	0
	4	- [illegible]	- [illegible]	0.540
	5	[illegible]	- [illegible]	0
	6	[illegible]	- [illegible]	0
23	1	- [illegible]	- 15[illegible]	1.[illegible]
	2	[illegible]	[illegible]	0
	3	[illegible]	- [illegible]	0
	4	- [illegible]	- [illegible]	0.540
	5	[illegible]	- [illegible]	0
	6	[illegible]	- [illegible]	0
24	1	- 54.225	[illegible]	[illegible]
	2	[illegible]	[illegible]	[illegible]
	3	- [illegible]	- [illegible]	[illegible]
	4	- [illegible]	[illegible]	[illegible]
	5	[illegible]	- [illegible]	[illegible]
	6	[illegible]	- [illegible]	[illegible]
25	1	- [illegible]	[illegible]	[illegible]
	2	- [illegible]	[illegible]	[illegible]
	3	[illegible]	- [illegible]	[illegible]
	4	- [illegible]	- [illegible]	[illegible]
	5	[illegible]	- [illegible]	[illegible]
	6	[illegible]	- [illegible]	[illegible]
26	1	- [illegible]	- [illegible]	[illegible]
	2	[illegible]	- [illegible]	[illegible]
	3	[illegible]	- [illegible]	[illegible]
	4	- [illegible]	[illegible]	[illegible]
	5	[illegible]	- [illegible]	[illegible]
	6	[illegible]	- [illegible]	[illegible]

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD
27	1	-43.694	-43.694	1.031
	2	-8.922	-8.922	0.195
	3	6.738	-3.689	0
	4	-3.689	6.733	0
	5	2.228	-2.225	0
	6	3.344	-3.341	0
28	1	-97.742	-79.490	1.261
	2	-2.128	0.399	0
	3	-14.853	-10.210	0.195
	4	1.436	-3.101	0
	5	3.322	-3.592	0
	6	4.145	-4.647	0

FRAMES A-A, D-D

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD (UNIFORM)	LOAD (CONC @ MID)
37	1	-79.946	-115.042	1.059	18.226
	2	-46.196	-51.942	0.240	15.360
	3	-1.592	3.233	0	0
	4	4.628	-14.638	0	0
	5	11.959	-10.365	0	0
	6	16.475	-14.346	0	0
38	1	-113.320	-37.190	1.059	18.226
	2	3.857	-3.116	0	0
	3	-49.768	-51.554	0.240	15.360
	4	-16.531	7.620	0	0
	5	8.694	-8.711	0	0
	6	12.214	-11.711	0	0
39	1	-31.233	-31.252	1.059	0
	2	-2.945	-3.002	0.240	0
	3	9.171	-21.773	0	0
	4	-22.459	9.823	0	0
	5	12.075	-12.003	0	0
	6	16.330	-16.758	0	0
40	1	-87.441	-112.645	1.059	18.226
	2	-2.229	3.049	0	0
	3	-52.010	-49.455	0.240	15.360
	4	7.398	-15.796	0	0
	5	8.297	-8.304	0	0
	6	11.740	-12.434	0	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD (UNIFORM)	LOAD (CONC.-MID)
41	1	-112.645	-57.441	1.059	18.220
	2	-49.406	-52.010	0.240	15.360
	3	3.099	-2.229	0	0
	4	-15.796	7.315	0	0
	5	8.752	-8.144	0	0
	6	12.350	-11.637	0	0
42	1	-31.252	-31.253	1.059	0
	2	9.823	-22.459	0	0
	3	-3.002	-2.745	0.240	0
	4	-21.773	3.172	0	0
	5	11.723	-11.740	0	0
	6	[illegible]	[illegible]	0	0
43	1	-87.190	-113.320	1.059	18.226
	2	-3.116	3.257	0	0
	3	7.620	-16.540	0	0
	4	-51.054	-49.768	0.240	15.360
	5	5.028	-8.326		0
	6	11.413	[illegible]		0
44	1	-116.042	-79.946	1.059	18.226
	2	3.233	-1.5[illegible]	0	0
	3	-14.[illegible]	4.[illegible]	0	0
	4	-51.942	-46.146	0.240	15.360
	5	9.750	-11.154	0	0
	6	13.797	-15.754	0	0
45	1	-87.242	-109.149	1.059	18.226
	2	-49.064	-52.404	0.240	15.360
	3	4.411	-14.510	0	0
	4	-2.324	[illegible]	0	0
	5	8.722	-9.[illegible]	0	0
	6	14.382	[illegible]	0	0
46	1	-108.791	-72.501	1.059	18.226
	2	-16.246	7.157	0	0
	3	-51.267	-51.[illegible]	0.24	15.360
	4	7.607	[illegible]	0	0
	5	7.036	-6.[illegible]	0	0
	6	11.473	-11.[illegible]	0	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD (UNIFORM)	LOAD (CONCENTRATED)
47	1	− 29.345	− 29.456	1.059	0
	2	− 24.345	13.777	0	0
	3	− 3.572	− 3.981	0.240	0
	4	13.145	− 23.661	0	0
	5	9.135	− 9.172	0	0
	6	14.309	− 14.171	0	0
48	1	− 92.327	− 108.317	1.059	18.226
	2	6.740	− 15.337	0	0
	3	− 51.979	− 50.936	0.240	15.360
	4	− 4.571	6.296	0	0
	5	[illegible]	− 7.080	0	0
	6	11.[illegible]	− 11.437	0	0
49	1	− 108.317	− 92.327	1.059	18.226
	2	− 50.436	− 51.979	0.240	15.360
	3	− 15.337	6.740	0	0
	4	6.296	− 4.571	0	0
	5	[illegible]	− 6.787	0	0
	6	[illegible]	− 10.925	0	0
50	1	− 29.[illegible]	− 29.[illegible]	1.059	0
	2	− 23.661	13.145	0	0
	3	− 3.981	− 3.572	0.240	0
	4	13.777	− 24.345	0	0
	5	[illegible]	− 9.107	0	0
	6	14.537	− 14.534	0	0
51	1	− 92.501	− 108.791	1.059	18.226
	2	7.107	− 16.246	0	0
	3	− 5.6[illegible]	7.307	0	0
	4	− 51.075	− 51.267	0.240	15.360
	5	6.742	− 6.[illegible]	0	0
	6	10.313	− 11.[illegible]	0	0
52	1	− 107.[illegible]	− 97.242	1.059	18.226
	2	− 52.401	− 49.064	0.240	15.360
	3	− 14.916	[illegible]	0	0
	4	6.053	− [illegible]	0	0
	5	7.039	− 3.586	0	0
	6	12.219	− 13.757	0	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD	LOAD
53	1	-90.133	-107.325	1.059	18.226
	2	-47.610	-51.520	0.240	[illegible]
	3	-2.020	5.009	0	0
	4	3.785	-14.431	0	0
	5	4.302	-4.272	0	0
	6	9.500	-5.556	0	0
54	1	-105.973	-95.027	1.059	18.226
	2	6.715	-5.311	0	0
	3	-50.553	-50.275	0.240	15.360
	4	-[illegible]	6.443	0	0
	5	4.206	-4.153	0	0
	6	3.213	-8.025	0	0
55	1	-29.044	-29.574	1.059	0
	2	-3.350	-3.417	0.240	0
	3	10.977	-23.076	0	0
	4	-23.220	11.310	0	0
	5	4.744	-4.750	0	0
	6	[illegible]	-9.377	0	0
56	1	-94.157	-107.001	1.059	18.226
	2	-4.303	5.517	0	0
	3	6.163	-14.324	0	0
	4	-50.340	-50.533	0.240	15.300
	5	4.270	-4.256	0	0
	6	7.914	-8.085	0	0
57	1	-107.001	-94.157	[illegible]	18.226
	2	-50.535	-50.340	0.240	15.300
	3	-14.324	6.163	0	0
	4	5.517	-4.303	0	0
	5	4.205	-4.189	0	0
	6	3.237	-7.325	0	0
58	1	-29.574	-29.044	1.059	0
	2	11.310	-23.220	0	0
	3	-23.076	10.977	0	0
	4	-3.417	-3.350	0.240	0
	5	4.907	-4.909	0	0
	6	9.220	-9.157	0	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD (UNIFORM)	LOAD (CONC'N'D)
59	1	− 45.027	−105.973	1.0[illegible]	[illegible].326
	2	− 5.391	6.71[illegible]	[illegible]	0
	3	6.443	− 16.[illegible]	[illegible]	0
	4	− 50.275	− 50.[illegible]3	0.240	15.360
	5	4.212	− 4.2[illegible]	0	0
	6	7.784	− [illegible].125	0	0
60	1	−107.825	− [illegible]0.133	1.059	1[illegible].326
	2	5.009	− 2.020	0	0
	3	− 14.431	3.735	0	0
	4	− 51.520	− 47.[illegible]	0.240	15.360
	5	4.[illegible]	− 4.143	0	0
	6	5.225	− 9.274	0	0
61	1	− 56.566	− 95.722	0.802	16.350
	2	− 17.236	− 13.192	0.090	5.760
	3	1.207	− 7.924	0	0
	4	− 1.325	4.470	0	0
	5	1.[illegible]	− 1.310	0	0
	6	4.345	− 3.467	0	0
62	1	− 92.247	− [illegible].540	0.802	16.350
	2	− 1[illegible].708	[illegible]	0	0
	3	− 1[illegible]	− [illegible]	0.090	5.760
	4	[illegible]	− [illegible]	0	0
	5	[illegible]	− [illegible]	0	0
	6	[illegible].714	− 3.992	0	0
63	1	− 31.443	− 32.741	0.802	0
	2	− 10.777	− 1[illegible].664	0.090	0
	3	7.614	− 4.516	0	0
	4	− 4.352	7.594	0	0
	5	1.081	− 1.111	0	0
	6	2.955	− 2.973	0	0
64	1	− 69.972	− 91.717	0.802	16.350
	2	1.747	− 9.357	0	0
	3	− 17.143	− 16.462	0.090	5.760
	4	− 4.520	5.063	0	0
	5	1.792	− 1.619	0	0
	6	3.779	− 3.514	0	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD (UNIFORM)	LOAD (CONC[illegible])
65	1	- 91.717	- 69.972	0.802	16.350
	2	- 16.462	- 17.143	0.090	5.760
	3	- 9.357	1.747	0	0
	4	5.063	- 4.520	0	0
	5	1.633	- 1.[illegible]	0	0
	6	3.485	- 3.727	0	0
66	1	- 32.741	- 31.443	0.302	0
	2	- 10.654	- 10.777	0.090	0
	3	7.394	- 4.3[illegible]2	0	0
	4	- 4.518	7.[illegible]24	0	0
	5	1.1[illegible]2	- 1.175	0	0
	6	[illegible].9[illegible]5	- 2.873	0	0
67	1	- 70.340	- 12.247	0.802	16.350
	2	- 5.407	6.475	0	0
	3	- 17.223	- 16.[illegible]	0.090	5.760
	4	2.[illegible]2	- 1[illegible].7[illegible]8	0	0
	5	1.[illegible]	- 1.763	0	0
	6	[illegible].163	- [illegible]	0	0
68	1	- 95.722	- 96.5[illegible]	0.802	16.350
	2	- 7.424	1.[illegible]	0	0
	3	- 16.192	- 17.2[illegible]	0.[illegible]	5.760
	4	4.476	- 1.325	0	0
	5	1.502	- 1.[illegible]	0	0
	6	3.273	- 4.074	0	0

FRAMES B-B, C-C

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD (UNIFORM)	LOAD (CONC[illegible])
37	1	- 54.908	- 87.049	0.431	18.220
	2	- 45.230	- 52.431	0.240	15.360
	3	- 2.002	3.970	0	0
	4	5.919	- 19.152	0	0
	5	15.170	- 12.756	0	0
	6	15.221	- 12.842	0	0
38	1	- 99.788	- 30.560	0.623	22.678
	2	4.[illegible]8	- 4.930	0	0
	3	- 23.[illegible]	10.227	0	0
	4	- [illegible]	- 68.[illegible]	0.480	16.240
	5	12.626	- 11.454	0	0
	6	12.716	- 11.772	0	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD [illegible]	LOAD [illegible]
39	1	− 33.227	− [illegible]	[illegible]	[illegible]
	2	− 5.0[illegible]	− 5.[illegible]	[illegible]	[illegible]
	3	15.054	− 3[illegible]	[illegible]	[illegible]
	4	− 37.570	13.612	0	[illegible]
	5	17.561	− 17.[illegible]	0	[illegible]
	6	17.7[illegible]	− 17.706	0	[illegible]
40	1	− 78.609	−105.420	0.628	22.6[illegible]
	2	− 3.775	4.521	0	0
	3	11.147	− 26.6[illegible]	0	0
	4	− [illegible]	− 61.3[illegible]	0.480	18.240
	5	11.347	− 12.453	[illegible]	0
	6	11.[illegible]	− 12.766	[illegible]	0
41	1	−105.[illegible]	− 78.709	0.628	22.693
	2	− 26.539	11.033	[illegible]	0
	3	4.485	− 3.652	[illegible]	0
	4	− 11.126	− 6[illegible]	0.480	18.240
	5	12.403	− 11.[illegible]	[illegible]	0
	6	12.727	− [illegible]		0
42	1	− 32.318	− 32.[illegible]	0.[illegible]	0
	2	13.791	− [illegible]	[illegible]	0
	3	− 5.244	− 5.603	[illegible]	0
	4	− 3[illegible]	12.[illegible]	0	[illegible]
	5	17.307	− 17.[illegible]	[illegible]	0
	6	17.616	− [illegible]	0	0
43	1	− 13.302	−106.433	0.628	22.6[illegible]
	2	− 5.510	[illegible]	0	[illegible]
	3	11.[illegible]	− 23.3[illegible]	0	[illegible]
	4	− 67.150	− [illegible]	0.480	18.2[illegible]
	5	10.450	− 11.631	0	0
	6	11.[illegible]	− 12.613	0	0
44	1	−10[illegible]	− 65.713	0.628	22.[illegible]
	2	4.541	− 2.15[illegible]	0	0
	3	− 22.657	6.[illegible]	0	0
	4	− 68.553	− 57.864	0.480	18.2[illegible]
	5	14.[illegible]	− 10.306	0	0
	6	14.567	− 17.257	0	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD [illegible]	LOAD [illegible]
45	1	−60.885	−81.656	0.628	22.6[illegible]
	2	−48.450	−52.370	0.240	15.[illegible]
	3	5.441	−10.150	0	0
	4	−2.304	7.442	0	0
	5	10.124	−9.4[illegible]	0	0
	6	11.192	−11.506	0	0
46	1	−96.477	−35.260	0.628	22.[illegible]
	2	−25.520	9.751	0	0
	3	−60.5[illegible]	−[illegible].005	0.480	18.2[illegible]
	4	9.346	−8.500	0	0
	5	10.124	−[illegible]	0	0
	6	11.[illegible]	−11.100	0	0
47	1	−31.070	−30.644	0.628	0
	2	−40.311	11.752	0	0
	3	−5.882	−7.312	0.480	0
	4	19.111	−39.232	0	0
	5	13.029	−13.062	0	0
	6	15.258	−15.275	0	0
48	1	−24.[illegible]	−[illegible].700	0.628	22.628
	2	9.[illegible]	−[illegible]	0	0
	3	−65.5[illegible]	−22.[illegible]	0.480	18.240
	4	−7.777	9.221	0	0
	5	[illegible]	−[illegible]	0	0
	6	10.477	−[illegible]	0	0
49	1	−100.804	−[illegible]	0.628	22.613
	2	−62.10[illegible]	−[illegible].549	0.480	18.240
	3	−27.546	[illegible].212	0	0
	4	9.151	−7.327	0	0
	5	[illegible].305	−9.286	0	0
	6	11.564	−10.407	0	0
50	1	−30.745	−20.181	0.628	0
	2	−[illegible].489	10.445	0	0
	3	−6.774	−6.[illegible]	0.480	0
	4	19.644	−[illegible]	0	0
	5	13.000	−13.010	0	0
	6	15.117	−15.109	0	0

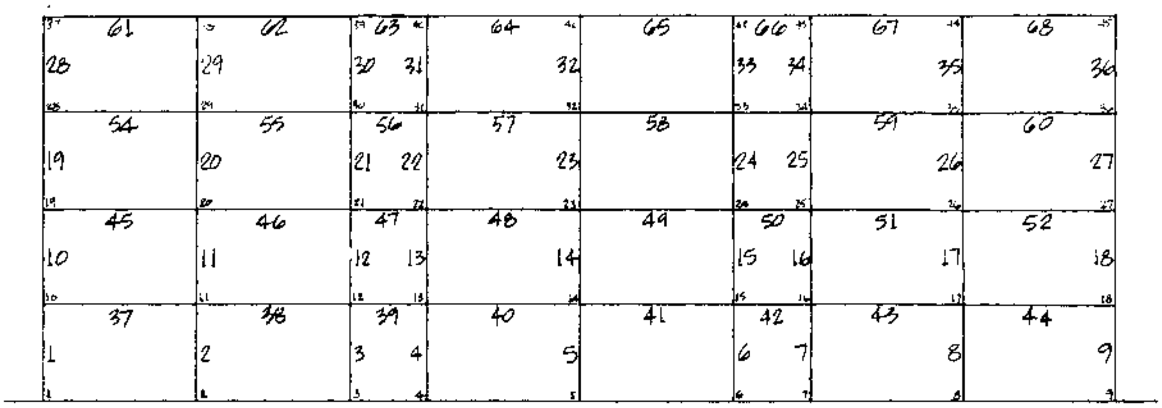
BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD (UNIFORM)	LOAD (CONC. M.D)
51	1	- 84.205	-101.010	0.628	22.695
	2	11.372	- 30.704	0	0
	3	- 9.903	11.757	0	0
	4	- 67.352	- 61.935	0.480	18.240
	5	9.259	- 9.611	0	0
	6	10.796	- 11.249	0	0
52	1	-102.679	- 76.672	0.628	22.695
	2	- 33.836	- 32.555	0.480	18.240
	3	- 22.390	5.653	0	0
	4	8.558	- 3.029	0	0
	5	1.333	- 12.518	0	0
	6	12.617	- 14.651	0	0
53	1	- 65.623	- 79.425	0.431	18.226
	2	- 8.580	6.171	0	0
	3	4.524	- 10.631	0	0
	4	- 41.603	- 51.585	0.240	15.360
	5	5.420	- 5.110	0	0
	6	8.739	- 7.410	0	0
54	1	- 91.913	- 88.466	0.628	22.698
	2	9.005	- 8.580	0	0
	3	- 24.159	9.137	0	0
	4	- 60.245	- 67.284	0.480	18.240
	5	6.121	- 5.981	0	0
	6	8.625	- 8.188	0	0
55	1	- 31.524	- 30.703	0.628	0
	2	- 5.859	- 6.182	0.480	0
	3	15.338	- 37.924	0	0
	4	- 38.256	15.558	0	0
	5	6.458	- 6.457	0	0
	6	9.367	- 9.310	0	0
56	1	- 56.416	- 98.327	0.628	22.698
	2	- 7.461	8.133	0	0
	3	9.495	- 25.775	0	0
	4	- 67.432	- 62.429	0.480	18.240
	5	5.920	- 5.942	0	0
	6	8.012	- 8.[illegible]	0	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD (UNIFORM)	LOAD (CONCENTRATED)
57	1	- 78.393	- 86.447	0.628	22.698
	2	- 25.881	9.300	0	0
	3	8.142	- 9.355	0	0
	4	- 62.432	- 67.500	0.480	18.240
	5	5.511	- 5.831	0	0
	6	8.201	- 7.952	0	0
58	1	- 30.841	- 30.771	0.628	0
	2	15.529	- 31.775	0	0
	3	- 31.528	15.272	0	0
	4	- 5.938	- 6.44[illegible]	0.480	0
	5	6.534	- 6.532	0	0
	6	9.266	- 9.260	0	0
59	1	- 37.275	- 97.443	0.628	22.698
	2	- 9.370	10.326	0	0
	3	10.427	- 10.021	0	0
	4	- 66.751	- 91.719	0.480	22.698
	5	5.379	- 5.[illegible]	0	0
	6	7.741	- 8.[illegible]	0	0
60	1	- 91.647	- 81.891	0.628	22.698
	2	7.[illegible]	- 2.818	0	0
	3	- 22.009	[illegible]	0	0
	4	- 81.317	- 66.497	0.480	18.240
	5	6.075	- 7.207	0	0
	6	8.422	- 9.850	0	0
61	1	- 50.579	- 49.321	0.628	20.120
	2	- 14.435	- 23.344	0.180	6.840
	3	1.360	- 20.762	0	0
	4	- 7.768	6.484	0	0
	5	2.035	- 1.[illegible]	0	0
	6	4.059	- 3.[illegible]	0	0
62	1	- 98.248	- 64.523	0.628	20.120
	2	- 16.226	5.740	0	0
	3	- 20.651	- 22.312	0.180	6.840
	4	3.665	- 3.[illegible]	0	0
	5	2.346	- 2.[illegible]	0	0
	6	3.317	- 4.030	0	0

BEAM #	LOAD CONDITION	LEFT END MOMENT	RIGHT END MOMENT	LOAD ([illegible])	LOAD ([illegible])
63	1	-36.315	-39.897	0.628	0
	2	-17.315	-18.[illegible]	0.180	0
	3	[illegible]	-[illegible].681	0	0
	4	-[illegible]	11.5[illegible]	0	0
	5	1.1[illegible]	-1.161	0	0
	6	2.535	-2.554	0	0
64	1	-71.106	-93.423	0.628	20.120
	2	3.452	-13.2[illegible]	0	0
	3	-8.625	7.625	0	0
	4	-22.577	-21.171	0.180	6.3[illegible]
	5	2.612	-2.200	0	0
	6	3.752	-3.491	0	0
65	1	-93.437	-71.707	0.628	20.120
	2	-21.2[illegible]	-22.56[illegible]	0.180	6.340
	3	7.5[illegible]	-5.373	0	0
	4	-[illegible]	3.471	0	0
	5	2.21[illegible]	-[illegible]	0	0
	6	3.474	-[illegible]	0	0
66	1	-39.415	-38.325	0.628	0
	2	-17.[illegible]7	-18.[illegible]	0.180	0
	3	[illegible]	-[illegible]	0	0
	4	-7.504	[illegible]	0	0
	5	1.2[illegible]	-[illegible]	0	0
	6	2.489	-2.4[illegible]	0	0
67	1	-71.440	-[illegible]	0.628	20.120
	2	-9.33[illegible]	[illegible].221	0	0
	3	-22.374	-20.7[illegible]	0.180	6.340
	4	5.037	-[illegible]	0	0
	5	2.784	-[illegible]	0	0
	6	[illegible]	-[illegible]	0	0
68	1	-93.474	-92.5[illegible]	0.628	20.120
	2	-10.652	1.030	0	0
	3	-23.051	-[illegible]	0.180	6.340
	4	[illegible]	-9.3[illegible]	0	0
	5	1.3[illegible]	-[illegible]	0	0
	6	3.04[illegible]	-[illegible]	0	0

COLUMN DESIGN

FRAMES A-A & D-D



DETERMINE IF FRAME IS BRACED:

WHEN THE STABILITY INDEX, $Q = (\Sigma P_u \Delta_u / H_u h_s)$, FOR A STORY IS NOT GREATER THAN 0.04, THE $P\Delta$ MOMENTS SHOULD NOT EXCEED 5 PERCENT OF THE FIRST ORDER MOMENTS AND THE STRUCTURE CAN BE CONSIDERED BRACED.

Q = STABILITY INDEX
P_u = FACTORED AXIAL LOAD
Δ_u = LATERAL DEFLECTION DUE TO H_u
H_u = TOTAL FACTORED LATERAL LOAD
h_s = STORY HEIGHT

CHECK THE FOLLOWING EQUATIONS SINCE THEY PRODUCE THE $Q_{MAXIMUM}$

1) $U = 0.75(1.4D + 1.7L + 1.7W)$
2) $U = 0.75(1.4D + 1.7L + 1.87E)$

FRAMES A-A, D-D

FIRST FLOOR COLUMNS (KIPS)

MBR. #	FRAMES A & D P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAMES A & D P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
1	146.835	173.179	320.014	144.616	173.179	317.795
2	303.776	155.342	459.118	304.151	155.342	459.493
3	179.623	155.387	335.010	176.946	155.387	332.333
4	187.012	155.387	342.399	189.720	155.387	345.107
5	300.914	155.387	456.301	300.939	155.387	456.326
6	180.106	155.387	335.493	177.511	155.387	332.898
7	186.565	155.387	341.952	189.196	155.387	344.583
8	302.936	155.387	458.323	302.638	155.387	458.025
9	152.182	170.284	322.466	154.230	170.284	324.514
ΣP_u			3371.076			3371.074

Δ_u EQ 1 = (1.7)(0.020 IN) = 0.03400 IN.

Δ_u EQ 2 = (1.87)(0.025 IN) = 0.04675 IN.

H_u EQ 1 = $(1.7)(2.964^K + 4.160^K + 3.786^K + 3.220^K) = 24.133^K$

H_u EQ 2 = $(1.87)(7.035^K + 5.695^K + 3.798^K + 1.898^K) = 34.457^K$

$h_s = (14 \text{ FT})(12 \text{ IN/FT}) = 168 \text{ IN.}$

$$Q_{EQ1} = \frac{(3371.076^K)(0.03400 \text{ IN})}{(24.133^K)(168 \text{ IN.})} = 0.023 < 0.04$$

$$Q_{EQ2} = \frac{(3371.074^K)(0.04675 \text{ IN})}{(34.457^K)(168 \text{ IN.})} = 0.027 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

SECOND FLOOR COLUMNS (KIPS)

MBR #	FRAMES A & D P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAMES A & D P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
10	108.128	124.060	232.188	106.523	124.060	230.583
11	220.020	111.947	331.967	220.275	111.947	332.222
12	132.260	111.913	244.173	130.456	111.913	242.369
13	136.269	111.913	248.182	138.114	111.913	250.027
14	218.190	111.913	330.103	218.214	111.913	330.127
15	132.604	111.913	244.517	130.575	111.913	242.[illegible]
16	135.925	111.913	247.838	137.680	111.913	249.593
17	219.705	111.913	331.618	219.531	111.913	331.444
18	111.179	121.997	233.176	112.611	121.997	234.608
ΣP_u			2443.762			2443.761

Δ_u EQ. 1 = (1.7)(0.020 IN) = 0.03400 IN.
Δ_u EQ. 2 = (1.87)(0.031 IN) = 0.05797 IN.
H_u EQ. 1 = $(1.7)(2.964^k + 4.160^k + 3.786^k) = 18.547^k$
H_u EQ. 2 = $(1.87)(7.035^k + 5.695^k + 3.798^k) = 30.907^k$
h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(2443.762^k)(0.03400\ IN.)}{(18.547^k)(168\ IN)} = 0.027 < 0.04$$

$$Q_{EQ.2} = \frac{(2443.762^k)(0.05797\ IN.)}{(30.907^k)(168\ IN.)} = 0.027 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

THIRD FLOOR COLUMNS (KIPS)

MBR #	FRAMES A&D Pu EQ. 1	PERP. FRAME Pu EQ. 1	TOTAL Pu EQ. 1	FRAMES A&D Pu EQ. 2	PERP. FRAME Pu EQ. 2	TOTAL Pu EQ. 2
19	68.236	74.419	142.655	67.339	74.419	141.758
20	138.014	68.006	206.020	138.145	68.006	206.151
21	83.750	67.935	151.685	82.870	67.935	150.805
22	85.301	67.935	153.236	86.213	67.935	154.148
23	136.753	67.935	204.688	136.769	67.935	204.704
24	84.047	67.935	151.982	83.212	67.935	151.147
25	84.333	67.935	152.268	84.258	67.935	152.193
26	137.974	67.935	205.909	137.900	67.935	205.835
27	69.549	73.296	142.845	70.325	73.296	143.621
ΣP_u			1511.288			1510.362

Δ_u EQ. 1 = (1.7)(0.014 IN.) = 0.02380 IN.
Δ_u EQ. 2 = (1.87)(0.024 IN.) = 0.04488 IN.
H_u EQ. 1 = $(1.7)(2.964^k + 4.160^k) = 12.111^k$
H_u EQ. 2 = $(1.87)(7.035^k + 5.695^k) = 23.805^k$
h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(1511.288^k)(0.02380\ IN.)}{(12.111^k)(168\ IN.)} = 0.018 < 0.04$$

$$Q_{EQ.1} = \frac{(1510.362^k)(0.04488\ IN.)}{(23.805^k)(168\ IN.)} = 0.017 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

FOURTH FLOOR COLUMNS (KIPS)

MBR #	FRAMES A & D P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAMES A & D P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
28	27.796	24.538	52.334	27.495	24.538	52.033
29	56.479	23.870	80.349	56.519	23.870	80.389
30	34.423	23.787	58.210	34.223	23.787	58.010
31	34.789	23.787	58.576	35.004	23.787	58.791
32	55.605	23.787	79.392	55.612	23.787	79.399
33	34.670	23.787	58.457	34.484	23.787	58.271
34	34.529	23.787	58.316	34.717	23.787	58.504
35	56.521	23.787	80.308	56.507	23.787	80.294
36	28.131	24.595	52.726	28.380	24.595	52.975
ΣP_u			578.668			578.666

Δ_u EQ. 1 = (1.7)(0.006 IN) = 0.01020 IN.

Δ_u EQ. 2 = (1.87)(0.014 IN) = 0.02618 IN.

H_u EQ. 1 = (1.7)(2.964^K) = 5.039^K

H_u EQ. 2 = (1.87)(7.035^K) = 13.155^K

h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(578.668^K)(0.01020\ IN)}{(5.039^K)(168\ IN.)} = 0.007 < 0.04$$

$$Q_{EQ.2} = \frac{(578.666^K)(0.02618\ IN.)}{(13.155^K)(168\ IN)} = 0.007 < 0.04$$

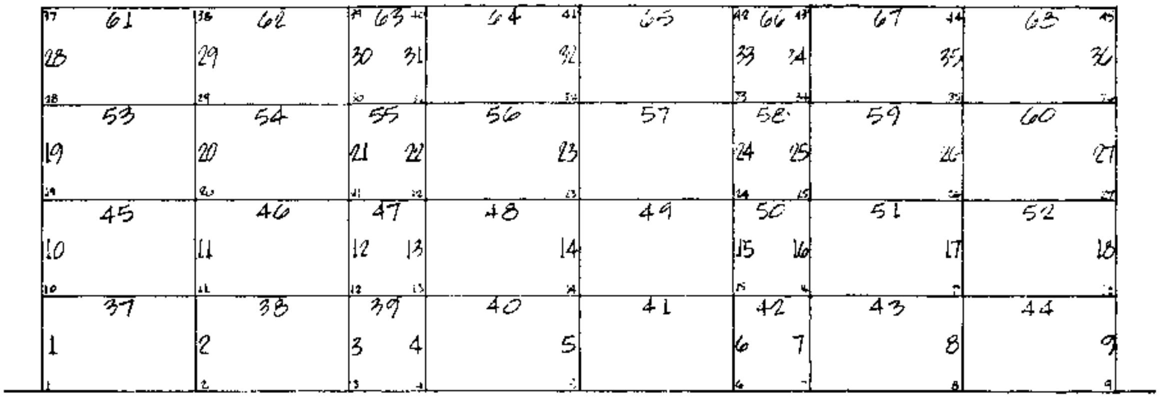
THE STORY CAN BE CONSIDERED BRACED

SINCE EVERY STORY CAN BE CONSIDERED BRACED, THE WHOLE FRAME CAN BE CONSIDERED BRACED

DETERMINE THE EFFECTIVE LENGTH FACTORS FOR EACH COLUMN USING THE JACKSON AND MORELAND ALIGNMENT CHARTS FOR BRACED FRAMES. USING A VALUE OF $0.5I_g$ FOR FLEXURAL MEMBERS (TO ACCOUNT FOR THE EFFECT OF CRACKING AND REINFORCEMENT ON RELATIVE STIFFNESS) AND I_g FOR COMPRESSION MEMBERS WHEN CALCULATING Ψ_A AND Ψ_B THE FOLLOWING K (EFFECTIVE LENGTH) FACTORS WERE DETERMINED.

MEMBER #	K-FACTOR	MEMBER #	K-FACTOR
28,36	0.89	21,22,24,25,12	0.82
29,32,35	0.82	13,15,16	
30,31,33,34	0.78	1,9	0.95
19,27,10,18	0.91	2,5,8	0.92
20,23,26,11,14,17	0.86	3,4,6,7	0.90

FRAMES B-B & C-C



DETERMINE IF THE FRAME CAN BE CONSIDERED BRACED

FRAMES B-B & C-C

FIRST FLOOR COLUMNS (KIPS)

MBR #	FRAMES B&C P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAMES B&C P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
1	123.407	248.653	372.060	122.332	248.653	370.985
2	291.080	225.450	516.530	291.146	225.450	516.596
3	184.948	197.644	382.592	183.575	197.644	381.219
4	193.696	197.644	391.340	195.110	197.644	392.754
5	320.672	197.644	518.316	320.688	197.644	518.332
6	183.872	197.644	381.516	182.520	197.644	380.164
7	193.264	197.644	390.908	194.637	197.644	392.281
8	324.190	197.644	521.834	324.009	197.644	521.653
9	160.140	225.614	385.754	161.256	225.614	386.870
ΣP_u			3860.850			3860.854

Δ_u EQ. 1 = (1.7)(0.025) = 0.04250 IN.

Δ_u EQ. 2 = (1.87)(0.023) = 0.04301 IN.

H_u EQ. 1 = $(1.7)(4.070^K + 5.720^K + 5.205^K + 4.519^K) = 33.184^K$

H_u EQ. 2 = $(1.87)(7.035^K + 5.695^K + 3.798^K - 1.898^K) = 34.457^K$

h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(3860.850^K)(0.04250\ IN.)}{(33.184^K)(168\ IN.)} = 0.029 < 0.04$$

$$Q_{EQ.2} = \frac{(3860.854^K)(0.04301\ IN.)}{(34.457^K)(168\ IN.)} = 0.029 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

SECOND FLOOR COLUMNS (KIPS)

MBR #	FRAMES B&C P_u EQ. 1	PERP. FRAMES P_u EQ. 1	TOTAL P_u EQ. 1	FRAMES B&C P_u EQ. 2	PERP. FRAMES P_u EQ. 2	TOTAL P_u EQ. 2
10	93.105	176.580	269.685	[illegible]	176.580	268.767
11	213.004	160.154	373.158	213.[illegible]	160.154	373.23[illegible]
12	135.316	141.689	[illegible]	135.201	141.689	276.890
13	141.234	141.689	[illegible]	143.[illegible]	141.689	[illegible]
14	232.234	141.689	373.923	[illegible]	141.689	373.[illegible]
15	136.067	141.689	277.756	134.988	141.689	276.677
16	140.724	141.689	282.413	141.821	141.689	[illegible]
17	234.949	141.689	376.638	234.805	141.689	376.494
18	117.059	161.399	278.458	117.[illegible]	161.399	27[illegible]
ΣP_u			2792.959			2792.957

Δ_u EQ. 1 = (1.7)(0.024 IN.) = 0.04080 IN.
Δ_u EQ. 2 = (1.87)(0.027 IN.) = 0.05049 IN.
H_u EQ. 1 = (1.7)($4.076^K + 5.720^K + 5.205^K$) = $25.50[illegible]^K$
H_u EQ. 2 = (1.87)($7.035^K + 5.695^K + 3.[illegible]^K$) = 30.907^K
l_{cs} = (14 FT)(12 IN./FT) = 168 IN.

$$Q_{EQ.1} = \frac{(2792.959^K)(0.04080\ IN.)}{(25.50[illegible]^K)(168\ IN.)} = 0.027 < 0.04$$

$$Q_{EQ.2} = \frac{(2792.957^K)(0.05049\ IN.)}{(30.907^K)(168\ IN.)} = 0.027 < 0.04$$

THEREFORE CAN BE CONSIDERED [illegible]

[illegible] FLOOR COLUMNS (KIPS)

MBR #	FRAMES B&C P_u EQ. 1	PERP. FRAMES P_u EQ. 1	TOTAL P_u EQ. 1	FRAMES B&C P_u EQ. 2	PERP. FRAMES P_u EQ. 2	TOTAL P_u EQ. 2
19	61.451	105.545	166.996	[illegible]	105.545	166.411
20	136.691	95.668	232.359	136.745	95.668	232.413
21	86.472	86.487	172.959	85.831	86.487	[illegible]
22	88.499	86.487	174.986	89.161	86.487	175.648
23	145.454	86.487	231.941	145.465	86.487	231.952
24	86.769	86.487	173.256	86.165	86.487	172.652
25	87.961	86.487	174.448	88.574	86.487	175.061
26	147.590	86.487	234.077	147.513	86.487	234.000
27	73.234	97.771	171.005	73.791	97.771	171.562
ΣP_u			1732.027			1732.025

Δ_u EQ. 1 = (1.7)(0.016 IN) = 0.02720 IN.
Δ_u EQ. 2 = (1.87)(0.021 IN) = 0.03927 IN.
H_u EQ. 1 = (1.7)($4.076^K + 5.720^K$) = 16.653^K
H_u EQ. 2 = (1.87)(7.025^K - 5.[illegible]^K) = 23.805^K
h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(1732.027^K)(0.02720\ IN.)}{(16.653^K)(168\ IN.)} = 0.017 < 0.04$$

$$Q_{EQ.2} = \frac{(1732.025^K)(0.03927\ IN.)}{(23.805^K)(168\ IN.)} = 0.017 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

FOURTH FLOOR COLUMNS

MBR #	FRAMES B&C P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAMES B&C P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
28	29.033	36.141	65.174	28.810	36.141	64.951
29	61.162	34.957	96.119	61.199	34.957	96.156
30	35.153	35.011	70.164	34.991	35.011	70.002
31	36.212	35.[illegible]	71.223	36.781	35.011	71.792
32	59.122	35.011	94.133	59.127	35.011	94.138
33	36.243	35.011	71.254	36.099	35.011	71.110
34	35.960	35.[illegible]	[illegible]	36.051	35.011	71.062
35	60.634	35.011	95.645	60.616	35.011	95.627
36	29.677	35.895	65.572	29.866	35.895	65.761
ΣP_u			700.601			700.599

Δ_u EQ. 1 = (1.7)(0.007 IN) = 0.01190 IN.
Δ_u EQ. 2 = (1.87)(0.012 IN) = 0.02244 IN.
H_u EQ. 1 = (1.7)(4.076^K) = 6.929^K
H_u EQ. 2 = (1.87)(7.035^K) = 13.155^K
h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(700.601^K)(0.01190\ IN.)}{(6.929^K)(168\ IN.)} = 0.007 < 0.04$$

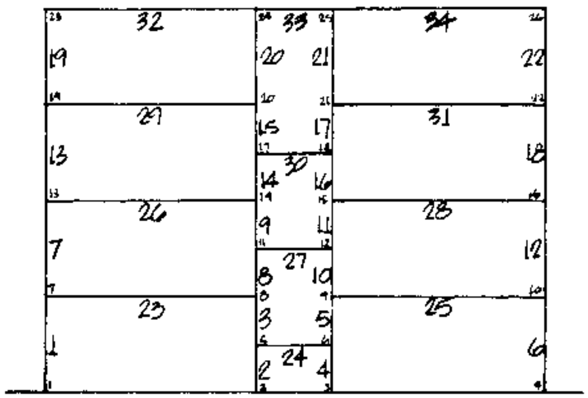
$$Q_{EQ.2} = \frac{(700.599^K)(0.02244\ IN.)}{(13.155^K)(168\ IN)} = 0.007 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED
SINCE EVERY STORY CAN BE CONSIDERED BRACED, THE WHOLE FRAME CAN BE CONSIDERED BRACED.

DETERMINE THE EFFECTIVE LENGTH FACTORS FOR EACH COLUMN USING THE PREVIOUSLY DESCRIBED METHOD.

MEMBER #	K-FACTOR	MEMBER #	K-FACTOR
28	0.87	23,26,14,17	0.83
29	0.79	27,18	0.90
30,31,33,34	0.81	1	0.94
32,35	0.79	2	0.91
36	0.86	3,4,6,7	0.89
19,10	0.91	5,8	0.91
20,11	0.84	9	0.94
21,22,24,25,12, 13,15,16	0.80		

II FRAME 1-1



DETERMINE IF THE FRAME CAN BE CONSIDERED BRACED

FRAME 1-1

FIRST FLOOR COLUMNS (KIPS)

MBR #	FRAME 1 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 2 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
1	175.162	145.611	320.773	174.889	145.611	320.500
2	244.262	123.517	367.779	244.541	123.517	368.058
4	265.317	123.517	388.834	265.051	123.517	388.568
6	182.200	145.611	327.811	182.458	145.611	328.069
ΣP_u			1405.197			1405.195

Δ_u EQ 1 = (1.7)(0.021 IN.) = 0.03570 IN.

Δ_u EQ 2 = (1.87)(0.016 IN.) = 0.02992 IN.

H_u EQ 1 = (1.7)($2.223^k + 3.120^k + 2.331^k +$ [illegible]$.465^k$) = 18.100^k

H_u EQ 2 = (1.87)($3.121^k + 2.527^k + 1.684^k + 0.342^k$) = 14.175^k

h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(1405.197^k)(0.03570\text{ IN.})}{(18.100^k)(168\text{ IN})} = 0.016 < 0.04$$

$$Q_{EQ.2} = \frac{(1405.197^k)(0.02992\text{ IN.})}{(14.175^k)(168\text{ IN.})} = 0.018 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

SECOND FLOOR COLUMNS (KIPS)

MBR #	FRAME 1 P_u EQ. 1	PERP FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 1 P_u EQ. 2	PERP FRAME P_u EQ. 2	TOTAL P_u EQ. 2
7	127.659	104.895	232.554	127.261	104.895	232.156
8	176.087	90.649	266.736	175.738	90.649	266.387
10	189.330	90.649	279.979	189.705	90.649	280.354
12	131.924	104.895	236.819	132.297	104.895	237.192
ΣP_u			1016.088			1016.089

Δ_u EQ. 1 = (1.7)(0.024 IN.) = 0.04080 IN.
Δ_u EQ. 2 = (1.87)(0.021 IN.) = 0.03927 IN.
H_u EQ. 1 = (1.7)(2.223^k + 3.120^k + 2.839^k) = 13.909^k
H_u EQ. 2 = (1.87)(3.121^k + 2.527^k + 1.684^k) = 13.711^k
h_s = (14 FT)(12 IN./FT) = 168 IN.

$$Q_{EQ1} = \frac{(1016.088^k)(0.04080\text{ IN})}{(13.909^k)(168\text{ IN.})} = 0.018 < 0.04$$

$$Q_{EQ2} = \frac{(1016.088^k)(0.03927\text{ IN.})}{(13.711^k)(168\text{ IN.})} = 0.017 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

THIRD FLOOR COLUMNS (KIPS)

MBR #	FRAME 1 P_u EQ. 1	PERP FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 2 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
13	79.180	63.520	142.700	78.844	63.520	142.364
14	108.658	57.119	165.777	108.141	57.119	165.260
16	113.965	57.119	171.084	114.500	57.119	171.619
18	81.260	63.520	144.780	81.579	63.520	145.099
ΣP_u			624.341			624.342

Δ_u EQ. 1 = (1.7)(0.017 IN) = 0.02890 IN.
Δ_u EQ. 2 = (1.87)(0.019 IN.) = 0.03553 IN.
H_u EQ. 1 = (1.7)(2.223^k + 3.120^k) = 9.083^k
H_u EQ. 2 = (1.87)(3.121^k + 2.527^k) = 10.562^k
h_s = (14 FT)(12 IN./FT) = 168 IN.

$$Q_{EQ1} = \frac{(624.341^k)(0.02890\text{ IN.})}{(9.083^k)(168\text{ IN.})} = 0.012 < 0.04$$

$$Q_{EQ2} = \frac{(624.342^k)(0.03553\text{ IN.})}{(10.562^k)(168\text{ IN})} = 0.013 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

FOURTH FLOOR COLUMNS

MBR #	FRAME 1 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 1 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
19	30.221	23.738	53.959	30.093	23.738	53.831
20	39.984	22.281	62.265	39.812	22.281	62.[illegible]
21	40.112	22.281	62.393	40.300	22.[illegible]	[illegible].581
22	30.815	23.738	54.553	30.926	23.738	54.664
ΣP_u			233.170			233.169

Δ_u EQ. 1 = (1.7)(0.011 IN.) = 0.01870 IN.
Δ_u EQ. 2 = (1.87)(0.014 IN.) = 0.02618 IN.
H_u EQ. 1 = (1.7)(2.223^K) = 3.779^K
H_u EQ. 2 = (1.87)(3.121^K) = 5.836^K
h_s = (14 FT.)(12 IN./FT.) = 168 IN.

$$Q_{EQ.1} = \frac{(233.170^K)(0.01870\ \text{IN})}{(3.779^K)(\text{[illegible] IN})} = 0.007 < 0.04$$

$$Q_{EQ.2} = \frac{(233.169^K)(0.02618\ \text{IN.})}{(5.836^K)(168\ \text{IN.})} = 0.006 < 0.04$$

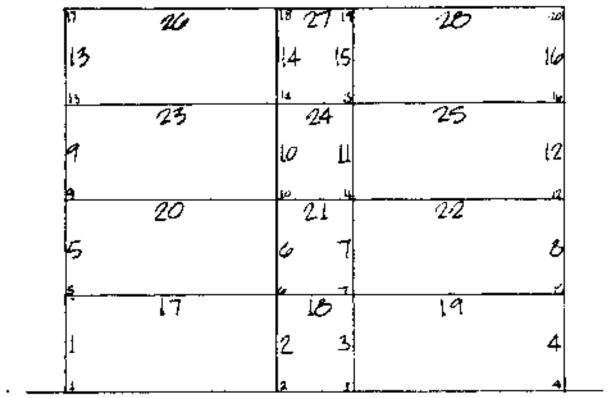
THE STORY CAN BE CONSIDERED BRACED

SINCE EVERY STORY CAN BE CONSIDERED BRACED, THE WHOLE FRAME CAN BE CONSIDERED BRACED.

DETERMINE THE EFFECTIVE LENGTH FACTORS FOR EACH COLUMN USING THE PREVIOUSLY DESCRIBED METHOD.

MEMBER #	K-FACTOR
19,22	0.87
20,21	0.83
13,18,7,12	0.91
15,17	0.92
14,16,8,10	0.93
9,11,3,5	0.93
1,6	0.95
2,4	0.91

FRAME 2-2



DETERMINE IF THE FRAME CAN BE CONSIDERED BRACED.

FRAME 2-2

FIRST FLOOR COLUMNS (KIPS)

MBR #	FRAME 2 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 2 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
1	159.974	297.442	451.416	157.377	297.442	454.819
2	212.489	285.424	497.913	219.529	285.424	504.953
3	241.967	285.424	527.391	234.929	285.424	520.353
4	168.502	297.442	465.944	165.096	297.442	462.538
ΣP_u			1942.664			1942.663

Δ_u EQ.1 $= (1.7)(0.059\text{ IN}) = 0.10030$ IN.

Δ_u EQ.2 $= (1.87)(0.023\text{ IN}) = 0.04301$ IN.

H_u EQ.1 $= (1.7)(4.446^K + 6.240^K + 5.670^K + 4.930^K) = 36.200^K$

H_u EQ.2 $= (1.87)(3.121^K + 2.527^K + 1.684^K + 0.842^K) = 15.285^K$

$h_s = (14\text{ FT})(12\text{ IN/FT}) = 168$ IN.

$$Q_{EQ.1} = \frac{(1942.664^K)(0.10030\text{ IN.})}{(36.200^K)(168\text{ IN.})} = 0.032 < 0.04$$

$$Q_{EQ.2} = \frac{(1942.663^K)(0.04301\text{ IN.})}{(15.285^K)(168\text{ IN.})} = 0.033 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

SECOND FLOOR COLUMNS (KIPS)

MBR #	FRAME 2 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 2 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
5	113.423	214.028	327.451	115.217	214.028	329.245
6	157.619	207.629	365.248	158.390	207.629	366.019
7	171.281	207.629	378.910	168.025	207.629	375.654
8	122.037	214.028	336.065	120.228	214.028	334.256
ΣP_u			1405.674			1405.674

Δ_u EQ. 1 = (1.7)(0.062 IN.) = 0.10540 IN.

Δ_u EQ. 2 = (1.87)(0.028 IN.) = 0.05236 IN.

H_u EQ. 1 = $(1.7)(4.446^k + 6.240^k + 5.678^k) = 27.819^k$

H_u EQ. 2 = $(1.87)(3.121^k + 2.527^k + 1.684^k) = 13.711^k$

h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(1405.674^k)(0.10540 \text{ IN.})}{(27.819^k)(168 \text{ IN.})} = 0.032 < 0.04$$

$$Q_{EQ.2} = \frac{(1405.674^k)(0.05236 \text{ IN.})}{(13.711^k)(168 \text{ IN.})} = 0.032 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

THIRD FLOOR COLUMNS (KIPS)

MBR #	FRAME 2 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 2 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
9	71.799	132.098	203.897	72.561	132.098	204.599
10	97.549	79.834	177.383	98.271	79.834	178.105
11	102.663	79.834	182.497	101.955	79.834	181.789
12	75.776	132.098	207.874	75.062	132.098	207.160
ΣP_u			771.651			771.653

Δ_u EQ. 1 = (1.7)(0.042 IN.) = 0.07140 IN.

Δ_u EQ. 2 = (1.87)(0.021 IN.) = 0.03927 IN.

H_u EQ. 1 = $(1.7)(4.446^k + 6.240^k) = 18.166^k$

H_u EQ. 2 = $(1.87)(3.121^k + 2.527^k) = 10.562^k$

h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(771.651^k)(0.07140 \text{ IN})}{(18.166^k)(168 \text{ IN})} = 0.018 < 0.04$$

$$Q_{EQ.2} = \frac{(771.653^k)(0.03927 \text{ IN})}{(10.562^k)(168 \text{ IN})} = 0.017 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

FOURTH FLOOR COLUMNS (KIPS)

MBR #	FRAME 2 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 2 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
13	29.234	51.413	80.647	29.395	51.413	80.808
14	36.591	56.415	93.006	36.516	56.415	92.931
15	37.049	56.415	93.464	37.131	56.415	93.546
16	30.347	51.413	81.760	30.179	51.413	81.592
ΣP_u			348.877			348.877

Δ_u EQ.1 = (1.7)(0.021 IN.) = 0.03570 IN.
Δ_u EQ.2 = (1.87)(0.013 IN.) = 0.02431 IN.
H_u EQ.1 = (1.7)(4.446^K) = 7.558^K
H_u EQ.2 = (1.87)(3.121^K) = 5.836^K
h_s = (14 FT.)(12 IN./FT.) = 168 IN.

$$Q_{EQ.1} = \frac{(348.877)(0.03570\text{ IN.})}{(7.558^K)(168\text{ IN.})} = 0.010 < 0.04$$

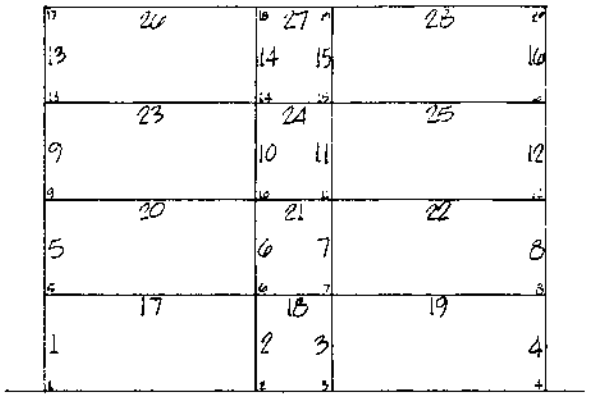
$$Q_{EQ.2} = \frac{(348.877)(0.02431\text{ IN.})}{(5.836^K)(168\text{ IN.})} = 0.009 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED
SINCE EVERY STORY CAN BE CONSIDERED BRACED, THE WHOLE FRAME CAN BE CONSIDERED BRACED.

DETERMINE THE EFFECTIVE LENGTH FACTORS FOR EACH COLUMN USING THE PREVIOUSLY DESCRIBED METHOD.

MEMBER #	K-FACTOR
13,16	0.87
14,15	0.75
9,12,5,8	0.89
10,11,6,7	0.78
1,4	0.94
2,3	0.87

FRAME 3-3



DETERMINE IF THE FRAME CAN BE CONSIDERED BRACED

FRAME 3-3

FIRST FLOOR COLUMNS (KIPS)

MBR #	FRAME 3 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 3 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
1	155.921	182.138	338.059	157.488	182.138	339.626
2	188.408	190.252	378.660	191.724	190.252	381.976
3	210.434	190.252	400.686	207.123	190.252	397.375
4	166.780	182.138	348.918	165.208	182.138	347.346
ΣP_u			1466.323			1466.323

Δ_u EQ. 1 $= (1.7)(0.044 \text{ IN.}) = 0.07480 \text{ IN.}$

Δ_u EQ. 2 $= (1.87)(0.023 \text{ IN.}) = 0.04301 \text{ IN.}$

H_u EQ. 1 $= (1.7)(3.335^k + 4.680^k + 4.259^k + 3.463^k) = 26.753^k$

H_u EQ. 2 $= (1.87)(3.121^k + 2.527^k + 1.684^k + 0.842^k) = 15.285^k$

$h_s = (14 \text{ FT})(12 \text{ IN./FT}) = 168 \text{ IN.}$

$$Q_{EQ.1} = \frac{(1466.323^k)(0.07480 \text{ IN.})}{(26.753^k)(168 \text{ IN.})} = 0.024 < 0.04$$

$$Q_{EQ.2} = \frac{(1466.323^k)(0.04301 \text{ IN.})}{(15.285^k)(168 \text{ IN.})} = 0.025 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

SECOND FLOOR COLUMNS (KIPS)

MBR #	FRAME 3 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 3 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
5	114.503	131.183	245.686	115.219	131.183	246.402
6	139.113	136.866	275.979	140.425	136.866	277.291
7	150.359	136.866	287.225	149.560	136.866	286.426
8	120.961	131.183	252.144	120.230	131.183	251.413
ΣP_u			1061.534			1061.532

Δ_u EQ. 1 = (1.7)(0.046 IN.) = 0.07820 IN.

Δ_u EQ. 2 = (1.87)(0.028 IN.) = 0.05236 IN.

H_u EQ. 1 = $(1.7)(3.335^k + 4.680^k + 4.259^k) = 20.866^k$

H_u EQ. 2 = $(1.87)(3.121^k + 2.527^k + 1.684^k) = 13.711^k$

h_s = (14 FT.)(12 IN./FT.) = 168 IN.

$$Q_{EQ.1} = \frac{(1061.534^k)(0.07820\text{ IN.})}{(20.866^k)(168\text{ IN.})} = 0.024 < 0.04$$

$$Q_{EQ.2} = \frac{(1061.534^k)(0.05236\text{ IN.})}{(13.711^k)(168\text{ IN.})} = 0.024 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

THIRD FLOOR COLUMNS (KIPS)

MBR #	FRAME 3 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 3 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
9	72.245	79.834	152.079	72.449	79.834	152.283
10	89.004	83.215	172.219	89.090	83.215	172.305
11	92.848	83.215	176.063	92.775	83.215	175.990
12	75.226	79.834	155.060	75.010	79.834	154.844
ΣP_u			655.421			655.422

Δ_u EQ. 1 = (1.7)(0.031 IN.) = 0.05270 IN.

Δ_u EQ. 2 = (1.87)(0.021 IN.) = 0.03927 IN.

H_u EQ. 1 = $(1.7)(3.335^k + 4.680^k) = 13.626^k$

H_u EQ. 2 = $(1.87)(3.121^k + 2.527^k) = 10.562^k$

h_s = (14 FT.)(12 IN./FT.) = 168 IN.

$$Q_{EQ.1} = \frac{(655.421^k)(0.05270\text{ IN.})}{(13.626^k)(168\text{ IN.})} = 0.015 < 0.04$$

$$Q_{EQ.2} = \frac{(655.421^k)(0.03927\text{ IN.})}{(10.562^k)(168\text{ IN.})} = 0.015 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

FOURTH FLOOR COLUMNS (KIPS)

MBR #	FRAME 3 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ 1	FRAME 3 P_u EQ. 2	PERP FRAME P_u EQ. 2	TOTAL P_u EQ. 2
13	29.278	31.204	60.482	29.300	31.204	60.504
14	36.737	30.998	67.735	36.606	30.998	67.604
15	37.084	30.998	68.082	37.221	30.998	68.219
16	30.112	31.204	61.316	30.084	31.204	61.288
ΣP_u			257.615			257.615

Δ_u EQ. 1 = (1.7)(0.015 IN.) = 0.02550 IN.
Δ_u EQ. 2 = (1.87)(0.013 IN.) = 0.02431 IN.
H_u EQ. 1 = (1.7)(3.335^K) = 5.670^K
H_u EQ. 2 = (1.87)(3.121^K) = 5.836^K
h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ.1} = \frac{(257.615^K)(0.02550\text{ IN.})}{(5.670^K)(168\text{ IN.})} = 0.007 < 0.04$$

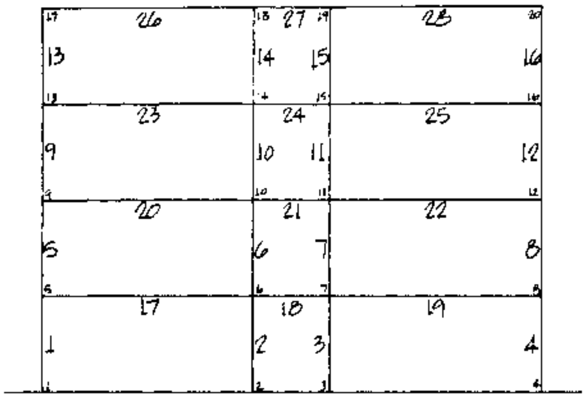
$$Q_{EQ.2} = \frac{(257.615^K)(0.02431\text{ IN.})}{(5.836^K)(168\text{ IN.})} = 0.006 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED
SINCE EVERY STORY CAN BE CONSIDERED BRACED, THE WHOLE FRAME CAN BE CONSIDERED BRACED.

DETERMINE THE EFFECTIVE LENGTH FACTORS FOR EACH COLUMN USING THE PREVIOUSLY DESCRIBED METHOD.

MEMBER #	K-FACTOR
13,16	0.87
14,15	0.75
9,12,5,8	0.89
10,11,6,7	0.78
1,4	0.94
2,3	0.87

⌗ FRAME 5-5



DETERMINE IF THE FRAME CAN BE CONSIDERED BRACED

FRAME 5-5

FIRST FLOOR COLUMN (KIPS)

MBR #	FRAME 5 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 5 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
1	154.085	295.203	449.288	157.488	295.203	452.691
2	184.684	315.621	500.305	191.724	315.621	507.345
3	214.162	315.621	529.783	207.123	315.621	522.744
4	168.614	295.203	463.817	165.208	295.203	460.411
ΣP_u			1943.193			1943.191

Δ_u EQ. 1 = (1.7)(0.059 IN.) = 0.10030 IN.

Δ_u EQ. 2 = (1.87)(0.023 IN.) = 0.04301 IN.

H_u EQ. 1 = $(1.7)(4.446^K + 6.240^K + 5.678^K + 4.930^K) = 36.200^K$

H_u EQ. 2 = $(1.87)(3.121^K + 2.527^K + 1.684^K + 0.842^K) = 15.285^K$

h_s = (14 FT)(12 IN/FT.) = 168 IN.

$$Q_{EQ.1} = \frac{(1943.193^K)(0.10030\text{ IN.})}{(36.200^K)(168\text{ IN.})} = 0.032 < 0.04$$

$$Q_{EQ.2} = \frac{(1943.193^K)(0.04301\text{ IN.})}{(15.285^K)(168\text{ IN.})} = 0.033 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

SECOND FLOOR COLUMNS (KIPS)

MBR #	FRAME 5 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 5 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
5	113.425	212.559	325.984	115.219	212.559	327.778
6	127.002	227.250	354.252	130.274	227.250	357.524
7	142.664	227.250	369.914	139.408	227.250	366.658
8	122.039	212.559	334.598	120.230	212.559	332.789
ΣP_u			1384.748			1384.749

Δ_u EQ. 1 $= (1.7)(0.062 \text{ IN.}) = 0.10540$ IN.
Δ_u EQ. 2 $= (1.87)(0.028 \text{ IN.}) = 0.05236$ IN.
H_u EQ. 1 $= (1.7)(4.446^k + 6.240^k + 5.678^k) = 27.819^k$
H_u EQ. 2 $= (1.87)(3.121^k + 2.527^k + 1.684^k) = 13.711^k$
$h_s = (14 \text{ FT})(12 \text{ IN./FT.}) = 168$ IN.

$$Q_{EQ.1} = \frac{(1384.748^k)(0.10540 \text{ IN.})}{(27.819^k)(168 \text{ IN.})} = 0.031 < 0.04$$

$$Q_{EQ.2} = \frac{(1384.749^k)(0.05236 \text{ IN.})}{(13.711^k)(168 \text{ IN.})} = 0.031 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

THIRD FLOOR COLUMNS (KIPS)

MBR #	FRAME 5 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 5 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
9	71.748	131.029	202.777	72.449	131.029	203.478
10	88.369	140.229	228.598	89.090	140.229	229.319
11	93.483	140.229	233.712	92.775	140.229	233.004
12	75.724	131.029	206.753	75.010	131.029	206.039
ΣP_u			871.840			871.840

Δ_u EQ. 1 $= (1.7)(0.041 \text{ IN}) = 0.06970$ IN.
Δ_u EQ. 2 $= (1.87)(0.021 \text{ IN}) = 0.03927$ IN.
H_u EQ. 1 $= (1.7)(4.446^k + 6.240^k) = 18.166^k$
H_u EQ. 2 $= (1.87)(3.121^k + 2.527^k) = 10.562^k$
$h_s = (14 \text{ FT})(12 \text{ IN/FT}) = 168$ IN.

$$Q_{EQ.1} = \frac{(871.840^k)(0.06970 \text{ IN.})}{(18.166^k)(168 \text{ IN.})} = 0.020 < 0.04$$

$$Q_{EQ.2} = \frac{(871.840^k)(0.03927 \text{ IN.})}{(10.562^k)(168 \text{ IN.})} = 0.019 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

FOURTH FLOOR COLUMNS

MBR. #	FRAME 5 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 5 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
13	29.139	50.253	79.392	29.300	50.253	79.553
14	36.685	54.191	90.876	36.610	54.191	90.801
15	37.143	54.191	91.334	37.225	54.191	91.416
16	30.252	50.253	80.505	30.084	50.253	80.337
ΣP_u			342.107			342.107

Δ_u EQ.1 = (1.7)(0.021 IN.) = 0.03570 IN.
Δ_u EQ.2 = (1.87)(0.013 IN.) = 0.02431 IN.
H_u EQ.1 = (1.7)(4.446^K) = 7.558^K
H_u EQ.2 = (1.87)(3.121^K) = 5.836^K
l_{15} = (14 FT.)(12 IN/FT.) = 168 IN.

$$Q_{EQ.1} = \frac{(342.107^K)(0.03570 \text{ IN.})}{(7.558^K)(168 \text{ IN.})} = 0.010 < 0.04$$

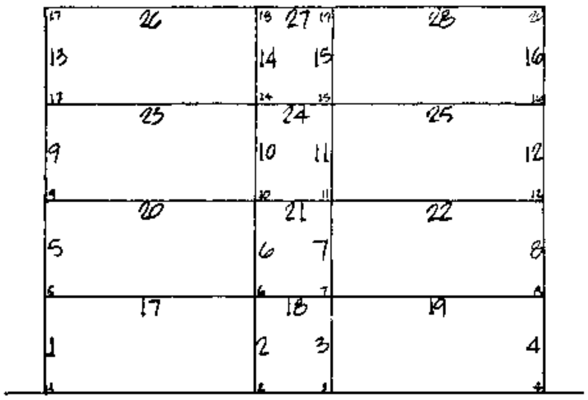
$$Q_{EQ.2} = \frac{(342.107^K)(0.02431 \text{ IN.})}{(5.836^K)(168 \text{ IN.})} = 0.008 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED
SINCE EVERY STORY CAN BE CONSIDERED BRACED, THE WHOLE FRAME CAN BE CONSIDERED BRACED

DETERMINE THE EFFECTIVE LENGTH FACTORS FOR EACH COLUMN USING THE PREVIOUSLY DESCRIBED METHOD.

MEMBER #	K-FACTOR
13,16	0.87
14,15	0.75
9,12,5,8	0.89
10,11,6,7	0.78
1,4	0.94
2,3	0.87

FRAME 9-9



DETERMINE IF THE FRAME CAN BE CONSIDERED BRACED

FRAME 9-9

FIRST FLOOR COLUMNS (KIPS)

MBR #	FRAME 9 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 5 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
1	172.776	145.611	318.387	172.542	145.611	318.153
2	222.018	153.487	375.505	221.669	153.487	375.156
3	236.855	153.487	390.342	237.219	153.487	390.706
4	179.943	145.611	325.554	180.162	145.611	325.773
ΣP_u			1409.788			1409.788

Δ_u EQ. 1 = (1.7)(0.032 IN.) = 0.05440 IN.

Δ_u EQ. 2 = (1.87)(0.025 IN.) = 0.04675 IN.

H_u EQ. 1 = $(1.7)(2.223^k + 3.120^k + 2.839^k + 2.465^k) = 18.100^k$

H_u EQ. 2 = $(1.87)(3.121^k + 2.527^k + 1.684^k + 0.842^k) = 15.285^k$

h_s = (14 FT)(12 IN/FT) = 168 IN.

$$Q_{EQ\,1} = \frac{(1409.788^k)(0.05440\text{ IN.})}{(18.100^k)(168\text{ IN.})} = 0.025 < 0.04$$

$$Q_{EQ\,2} = \frac{(1409.788^k)(0.04675\text{ IN.})}{(15.285^k)(168\text{ IN.})} = 0.026 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

SECOND FLOOR COLUMNS (KIPS)

MBR. #	FRAME 9 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 9 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
5	125.879	104.895	230.774	125.523	104.895	230.418
6	161.963	110.811	272.774	161.317	110.811	272.128
7	170.111	110.811	280.922	170.775	110.811	281.586
8	130.149	104.895	235.044	130.486	104.895	235.381
ΣP_u			1019.514			1019.513

Δ_u EQ.1 $= (1.7)(0.034\text{ IN.}) = 0.05780$ IN.
Δ_u EQ.2 $= (1.87)(0.031\text{ IN.}) = 0.05797$ IN.
H_u EQ.1 $= (1.7)(2.223^k + 3.120^k + 2.839^k) = 13.909^k$
H_u EQ.2 $= (1.87)(3.121^k + 2.527^k + 1.684^k) = 13.711^k$
$h_s = (14\text{ FT})(12\text{ IN./FT.}) = 168$ IN.

$$Q_{EQ.1} = \frac{(1019.514^k)(0.05780\text{ IN.})}{(13.909^k)(168\text{ IN.})} = 0.025 < 0.04$$

$$Q_{EQ.2} = \frac{(1019.513^k)(0.05797\text{ IN.})}{(13.711^k)(168\text{ IN.})} = 0.026 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

THIRD FLOOR COLUMNS (KIPS)

MBR #	FRAME 9 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 9 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
9	78.339	63.520	141.859	78.049	63.520	141.569
10	103.358	67.293	170.651	102.805	67.293	170.098
11	106.254	67.293	173.547	106.823	67.293	174.116
12	80.302	63.520	143.822	80.579	63.520	144.099
ΣP_u			629.879			629.882

Δ_u EQ.1 $= (1.7)(0.024\text{ IN.}) = 0.04080$ IN.
Δ_u EQ.2 $= (1.87)(0.025\text{ IN.}) = 0.04675$ IN.
H_u EQ.1 $= (1.7)(2.223^k + 3.120^k) = 9.083^k$
H_u EQ.2 $= (1.87)(3.121^k + 2.527^k) = 10.562^k$
$h_s = (14\text{ FT.})(12\text{ IN./FT.}) = 168$ IN.

$$Q_{EQ.1} = \frac{(629.879^k)(0.04080\text{ IN.})}{(9.083^k)(168\text{ IN.})} = 0.017 < 0.04$$

$$Q_{EQ.2} = \frac{(629.882^k)(0.04675\text{ IN.})}{(10.562^k)(168\text{ IN.})} = 0.017 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED

FOURTH FLOOR COLUMNS

MBR #	FRAME 9 P_u EQ. 1	PERP. FRAME P_u EQ. 1	TOTAL P_u EQ. 1	FRAME 9 P_u EQ. 2	PERP. FRAME P_u EQ. 2	TOTAL P_u EQ. 2
13	30.345	22.281	52.626	30.226	22.281	52.507
14	39.745	23.942	63.687	39.556	23.942	63.498
15	40.142	23.942	64.084	40.340	23.942	64.282
16	30.894	22.281	53.175	31.005	22.281	53.286
ΣP_u			233.572			233.573

Δ_u EQ.1 = (1.7)(0.012 IN) = 0.02040 IN.
Δ_u EQ.2 = (1.87)(0.015 IN) = 0.02805 IN.
H_u EQ.1 = (1.7)(2.223^K) = 3.779^K
H_u EQ.2 = (1.87)(3.121^K) = 5.836^K
h_s = (14 FT.)(12 IN/FT.) = 168 IN.

$$Q_{EQ1} = \frac{(233.572^K)(0.02040\text{ IN})}{(3.779^K)(168\text{ IN})} = 0.008 < 0.04$$

$$Q_{EQ2} = \frac{(233.572^K)(0.02805\text{ IN.})}{(5.836^K)(168\text{ IN.})} = 0.007 < 0.04$$

THE STORY CAN BE CONSIDERED BRACED
SINCE EVERY STORY CAN BE CONSIDERED BRACED, THE WHOLE FRAME CAN BE CONSIDERED BRACED.

DETERMINE THE EFFECTIVE LENGTH FACTORS FOR EACH COLUMN USING THE PREVIOUSLY DESCRIBED METHOD.

MEMBER #	K-FACTOR
13,16	0.90
14,15	0.77
9,12,5,8	0.91
10,11,6,7	0.79
1,4	0.94
2,3	0.88

II COLUMN DESIGN

DESIGN OF THE [illegible] COLUMNS [illegible] COLUMN INTERACTION [illegible]. THE [illegible] DIAGRAM [illegible] WITH THE [illegible] LOAD EQUATIONS:

[illegible]

LOAD EQUATIONS [illegible] WITH THREE TYPES [illegible] COMBINATIONS. [illegible] LOAD CONDITIONS [illegible]

TYPE 1 [illegible] LOAD.

TYPE 2 [illegible] IN THE MAIN FRAME WITH THE MINIMUM [illegible] AXIAL LOAD.

TYPE 3 [illegible] IN THE PERPENDICULAR FRAME [illegible]

[illegible]

MBR #'S COLUMN ARE [illegible] FRAME. THE CIRCLED NUMBER [illegible] COMBINATIONS. THE OTHER [illegible] OF THE [illegible].

COLUMN NAME	TYPE	MAIN FRAME		PERPENDICULAR FRAME	
		MBR #'S	L.L. CONDITIONS	MBR #'S	L.L. CONDITIONS
A1-1, D1-1	1	1 & 7	2,5 [illegible]	[illegible] & 6	2,3,5,6
	2		[illegible]		4, [illegible]
	3		3,4, [illegible]		3,4, [illegible]
A1-2, D1-2	[illegible]	[illegible] & [illegible]	2,5 [illegible]	[illegible] & 12	2,3,5,6
	[illegible]		[illegible]		[illegible],4, [illegible]
	[illegible]		[illegible]		2,5, [illegible]

COLUMN NAME	TYPE	MAIN FRAME		PERPENDICULAR FRAME	
		MPC #	[illegible]	[illegible]	[illegible]
A1-3, D1-3	1 2 3	(2) ≠ [illegible]	2,5,[illegible] 2,[illegible] [illegible]	[illegible]	2,3,4,5,6 [illegible] [illegible]
A1-4, D1-4	[illegible] [illegible] 3	(12) ≠ 2	2,6,7,[illegible] 2,3,6 2,3,4,5,6	[illegible]	2,4,5,6 2,5,7 2,4,6,7
A2-1, D2-1	1 2 3	(2) ≠ 3	2,3,4,[illegible] 2,4,[illegible] [illegible]	(1) ≠ 4	2,5,7 2,3,4,[illegible],7 [illegible]
A2-2, D2-2	[illegible] [illegible] 3	(11) ≠ 17	[illegible] 2,4,[illegible] 2,3,4,[illegible]	(5) ≠ 5	2,5,6 ½,3,4,½,7 2,3,4,5,7
A2-3, D2-3	[illegible]	(22) ≠ 22	2,3,4,[illegible] [illegible] [illegible]	[illegible]	2,3,5 ½,4,½,6,7 2,4,5,6
A2-4, D2-4	[illegible] [illegible] 3	(2) ≠ [illegible]	[illegible] [illegible] [illegible]	(12) ≠ [illegible]	[illegible] [illegible],4,7 [illegible]
A3-1, D3-1 A7-1, D7-1	1 2 3	3 ≠ (7)	[illegible] [illegible] [illegible]	(1) ≠ 4	2,5,7 ½,3,4,½,6 2,5,7
A3-2, D3-2 A7-2, D7-2	1 2 3	12 ≠ ([illegible])	[illegible],4,[illegible] 4,5,[illegible] [illegible]	(5) ≠ 5	2,5,6 ½,3,4,½,7 2,3,4,5,7
A3-3, D3-3 A7-3, D7-3	1 [illegible] [illegible]	21 ≠ (23)	2,3,4,[illegible] 2,4,[illegible] [illegible]	(7) ≠ [illegible]	2,3,5 ½,4,½,6,7 [illegible]
A3-4, D3-4 A7-3, D7-4	[illegible]	[illegible] ≠ (24)	[illegible],4,6,7 [illegible] [illegible]	([illegible]) ≠ 12	2,5,6 2,3,4,7 2,3,5,6,7
A4-1, D4-1 A8-1, D8-1	[illegible]	4 ≠ (6)	[illegible] [illegible] [illegible]	(1) ≠ 4	2,5,7 ½,3,4,½,6 [illegible]
A4-2, D4-2 A8-2, D8-2	1 2 3	13 ≠ (15)	[illegible] 2,3,5 2,3,4,[illegible]	(5) ≠ 5	2,5,6 ½,3,4,½,7 [illegible]

COLUMN NAME	TYPE	[illegible]		[illegible]	
		[illegible]	[illegible]	[illegible]	[illegible]
A4-[illegible], [illegible] [illegible]	1 2 3	[illegible]	[illegible] 2,4,[illegible] [illegible]	[illegible]	2,5,[illegible] [illegible] [illegible]
[illegible]	1 2 3	[illegible]	[illegible] 2,[illegible] [illegible]	[illegible]	[illegible] [illegible] [illegible]
A5-1, D4-1	[illegible] [illegible] 3	[illegible]	2,5,[illegible] 2,4,[illegible] [illegible]	1 [illegible] 4	2,5,7 [illegible] [illegible]
A5-2, [illegible]	[illegible]	4	[illegible] [illegible] [illegible]	5 & 5	2,5,6 [illegible],3,4,[illegible],7 [illegible]
A5-3, [illegible]	[illegible]	23	2,5,6,9 2,4,[illegible],7 [illegible]	9 & 12	[illegible] [illegible],4,[illegible],6,7 2,4,[illegible]
A5-4, [illegible]-4	[illegible]	[illegible]	2,3,4,5 [illegible] [illegible]	[illegible]	[illegible] [illegible],3,4,[illegible] [illegible]
A8-1, [illegible]-1	1 2 3	2 & 5	3,4,[illegible] [illegible] [illegible]	1 & 4	2,5,7 [illegible],3,4,[illegible],6 [illegible]
[illegible]-2, [illegible]-2	1 2 3	11 & 17	[illegible] [illegible] [illegible]	[illegible]	2,5,6 [illegible] [illegible]
[illegible]	[illegible]	20 & 26	[illegible] [illegible] [illegible]	9 & [illegible]	[illegible] [illegible],4,[illegible],6,7 2,4,[illegible]
[illegible]-4, [illegible]-4	[illegible]	29 & [illegible]	[illegible] [illegible] [illegible]	13 & [illegible]	[illegible] [illegible],3,4,7 [illegible]
A9-1, [illegible]-1	[illegible]	4 & 9	[illegible] [illegible] [illegible]	1 & 4	2,4,[illegible] [illegible] 3,5,7
A9-2, [illegible]-2	[illegible]	[illegible] & 13	[illegible] [illegible] [illegible]	5 & [illegible]	[illegible],4,5,6 [illegible],7 [illegible]

COLUMN NAME	TYPE	MAIN FRAME		PERPENDICULAR FRAME	
		BAR #'s	L.L. LOCATION	BAR #'s	L.L. LOCATION
A9-3, D9-3	1	19 & 27	2,5,6,9	9 & 12	2,3,5
	2		2,5,8,9		2,4,5,6,7
	3		3,4,5,7,8,9		2,4,5,6
A9-4, D9-4	1	28 & 36	5,6,8,9	13 & 16	2,4,5,6
	2		4,5,9		2,3,7
	3		2,3,4,5,7		2,3,5,7
B1-1a, C1-1a	1	1 & 9	2,5,6,9	2 & 4	2,3,4,5,6,7
	2		3,6,8,9		5
	3		2,3,4,7,8		2,6,7
B1-1b, C1-1b	1	1 & 9	2,5,6,9	3 & 5	2,3,4,5,6,7
	2		3,6,8,9		5
	3		3,4,6,7,8		5,6,7
B1-2a, C1-2a	1	10 & 18	2,5,6,9	3 & 10	2,3,4,5,6,7
	2		2,5,6		2,3,5
	3		2,3,4,6,7,8		2,3,5,7
B1-2b, C1-2b	1	10 & 18	2,5,6,9	9 & 11	2,3,5,6,7
	2		2,5,6		2,3,4,5
	3		2,3,4,7,8		3,6,7
B1-3a, C1-3a	1	19 & 27	2,5,6,9	4 & 16	2,3,5,6,7
	2		[illegible],6,7,9		2,3,4,5
	3		2,3,4,6,7,8		2,4,5,6
B1-3b, C1-3b	1	19 & 27	2,5,6,9	15 & 17	2,3,5,7
	2		2,5,6,7,9		2,3,4,5,6
	3		2,3,4,6,7,8		2,5,6,7
B1-4, C1-4	1	28 & 36	2,6,7,9	20 & 21	2,3,7
	2		2,3,6		2,3,4,5,6
	3		2,3,4,5,[illegible],6		2,3,4,5,6
B2-1, C2-1	1	2 & 8	2,3,4,6,7,8	2 & 3	2,3,5,6
	2		2,7,8,9		3/2,3/2,4,5/2,6/2,7
	3		2*,3*,5,6-,7+,7		3,5
B2-2, C2-2	1	11 & 17	2,3,6,7,8	6 & 7	2,3,5,6,7
	2		3,5,7,9		3/2,3/2,4,5/2,6/2
	3		2+,3+,4,5,6+,7+,9		2,5
B2-3, C2-3	1	20 & 26	2,3,4,6,7,8	10 & 11	2,3,4,5,6
	2		3,5,7		3/2,3/2,5/2,6/2,7
	3		2+,3+,5,6,7+,9		2,5

COLUMN NAME	TYPE	MAIN FRAME		PERPENDICULAR FRAME	
		MBR #'s	L.L. CONDITION	MBR. #'s	L.L. CONDITION
B2-4, C2-4	1	29 & 34	2,3,8	14 & 15	2,3,7
	2		3,7,9		3/2,3/2
	3		2+,3+,4,5,6+,7+		2,5
B3-1, C3-1	1	3 & 7	3,4,7,8	2 & 3	2,3,5,6
	2		2,4,7,9		3/2,3/2,4,[illegible]
	3		2,3+,4/2,5,7+,8/2,9		2,5
B3-2, C3-2	1	12 & 16	3,4,7,8,9	6 & 7	[illegible]
	2		2,3,7,9		3/2,3/2,4,5/2,6/2
	3		2,3+,4/2,5,6,7,8/2		2,5
B3-3, C3-3	1	21 & 25	3,4,5,7,8	10 & 11	2,3,4,5,6
	2		3,5,6,7		3/2,3/2,5/2,6/2
	3		2,3+,4/2,6,7+,8/2,9		2,5
B3-4, C3-4	1	30 & 34	3,4,6,9	14 & 15	2,3,7
	2		2,3,4,7		2,3,4,5,6
	3		2,3+,4/2,5,7+,8/2		2,5
B4-1, C4-1	1	4 & 6	4,5,8,9	2 & 3	2,3,5,6
	2		2,7,9		4,5/2,6/2,7
	3		2,3,4/2,5+,6,[illegible],8/2,9+		3,5
B4-2, C4-2	1	13 & 15	4,5,7,8,9	6 & 7	2,3,5,6,7
	2		5,7,9		2/2,3/2,4,5/2,6/2
	3		2,3,4/2,5+,6,8/2,9+		2,5
B4-3, C4-3	1	22 & 24	3,4,5,8,9	10 & 11	2,3,4,5,6
	2		3,5,6,9		3/2,3/2,5/2,6/2,7
	3		2,4/2,5+,[illegible],8/2,9+		2,5
B4-4, C4-4	1	31 & 33	4,5,6,7,8	14 & 15	2,3,7
	2		2,5,9		2/2,3/2,4,5/2,6/2
	3		2,3,4/2,5+,8/2,9+		2,5
B5-1, C5-1	1	5	2,5,6,9	2 & 3	2,3,5,6
	2		2,4,7,9		3/2,3/2,4,5/2,6/2,7
	3		2+,3,4,5+,6+,7,8,9+		2,5
B5-2, C5-2	1	14	2,5,6,9	6 & 7	2,3,5,6,7
	2		4,5,7,9		[illegible],4,[illegible]
	3		2+,3,4,5+,6+,7,8,9+		2,5
B5-3, C5-3	1	23	2,5,6,9	10 & 11	2,3,4,5,6
	2		2,6,7		3/2,3/2,[illegible],5/2,7
	3		2+,3,4,[illegible],7,8,9+		2,5

COLUMN NAME	TYPE	MAIN FRAME		PERPENDICULAR FRAME	
		MBR #'S	L.L. CONDITION	MBR #'S	L.L. CONDITION
B5-4, C5-4	1	32	2,5	14 & 15	2,3,7
	2		4,5,8,9		3/2,3/2,4,5/2,6/2
	3		2+,3,4,5+,6+,7,8,9+		2,5
B6-1, C6-1	1	6 & 4	2,3,6,7	2 & 3	2,3,5,6
	2		5,6,8		4,5/2,6/2,7
	3		2+,3/2,4,5,6+,7/2,8,9		3,5
B6-2, C6-2	1	15 & 13	2,3,6,7,8	6 & 7	2,3,5,6,7
	2		2,6,8		4/2,3/2,4,5/2,6/2
	3		2+,3/2,4,5,6+,7/2,9		2,5
B6-3, C6-3	1	24 & 22	2,3,4,6,7	10 & 11	2,3,4,5,6
	2		2,4,6,9		3/2,3/2,5/2,6/2,7
	3		2,3/2,5,6+,7/2,8,9		2,5
B6-4, C6-4	1	33 & 31	2,3,7,8,9	14 & 15	2,3,7
	2		2,5,6		3/2,3/2,4,5/2,6/2
	3		2+,3/2,4,5,6+,7/2		2,5
B7-1, C7-1	1	7 & 5	3,4,7,8	2 & 3	2,3,5,6
	2		5,7,9,8		3/2,3/2,4,5/2,6/2,7
	3		2,3/2,4+,5,6,7/2,8+,9		3,5
B7-2, C7-2	1	16 & 12	3,4,6,7,8	6 & 7	2,3,5,6,7
	2		4,5,6,8		3/2,3/2,4,5/2,6/2
	3		2,3/2,4+,5,7/2,8+,9		2,5
B7-3, C7-3	1	25 & 21	2,3,4,7,8	10 & 11	2,3,4,5,6
	2		2,4,8,9		3/2,3/2,5/2,6/2,7
	3		3/2,4+,5,6,7/2,8+,9		2,5
B7-4, C7-4	1	34 & 30	3,4,6,9	14 & 15	2,3,7
	2		3,4,5,8		2,3,4,5/2,6/2
	3		2,3/2,4+,5,7/2,8+		2,5
B8-1, C8-1	1	8 & 2	3,4,5,7,8,9	2 & 3	2,3,5,6
	2		2,3,4,9		3/2,3/2,4,5/2,6/2
	3		2,4+,5+,6,8+,9+		3,5
B8-2, C8-2	1	17 & 11	4,5,7,8,9	6 & 7	2,3,5,6,7
	2		5,7,9		3/2,3/2,4,5/2,6/2
	3		2,3,4+,5+,6,8+,9+		2,5
B8-3, C8-3	1	26 & 20	3,4,5,7,8,9	10 & 11	2,3,4,5,6
	2		3,5,6,9		3/2,3/2,5/2,6/2,7
	3		2,4+,5+,6,8+,9+		2,5

COLUMN NAME	TYPE	MAIN FRAME		PERPENDICULAR FRAME	
		MBR. #'S	L.L. CONDITION	MBR. #'S	L.L. CONDITION
B8-4, C8-4	1	35 & 29	$\dot{4},\dot{5},7$	14 & 15	$\dot{2},\dot{3},7$
	2		$2,3,\dot{5},6,7,\dot{9}$		$\dot{2}/2,\dot{3}/2,4,\dot{5}/2,\dot{6}/2$
	3		$2,3,\dot{4}+,\dot{5}+,6,\dot{8}+,\dot{9}+$		$\dot{2},\dot{5}$
B9-1, C9-1	1	9 & 1	$2,\dot{5},6,\dot{9}$	2 & 3	$\dot{2},3,5,6$
	2		$4,6,7,\dot{9}$		$4,\dot{5},\dot{6},7$
	3		$3,4,\dot{5}*,7,8,\dot{9}+$		$3,\dot{5}$
B9-2, C9-2	1	18 & 10	$2,\dot{5},6,\dot{9}$	6 & 7	$\dot{2},\dot{3},5,\dot{6},7$
	2		$2,3,\dot{5},9$		$\dot{2},\dot{3},4,\dot{5},\dot{6}$
	3		$3,4,\dot{5}+,7,8,\dot{9}+$		$\dot{2},5,7$
B9-3, C9-3	1	27 & 19	$2,\dot{5},6,\dot{9}$	10 & 11	$\dot{2},\dot{3},4,\dot{5},\dot{6}$
	2		$2,\dot{5},6,8,\dot{9}$		$\dot{2},\dot{3},5,6,7$
	3		$3,4,\dot{5}+,7,8,\dot{9}+$		$\dot{2},4,\dot{5}$
B9-4, C9-4	1	36 & 28	$\dot{5},6,8,\dot{9}$	14 & 15	$\dot{2},\dot{3},7$
	2		$4,\dot{5},\dot{9}$		$\dot{2},\dot{3},4,\dot{5},\dot{6}$
	3		$2,3,4,\dot{5}+,7,\dot{9}+$		$\dot{2},\dot{5}$

· INDICATES A LOAD CONDITION WHICH HAS A LOAD BAY ADJACENT TO THE COLUMN.

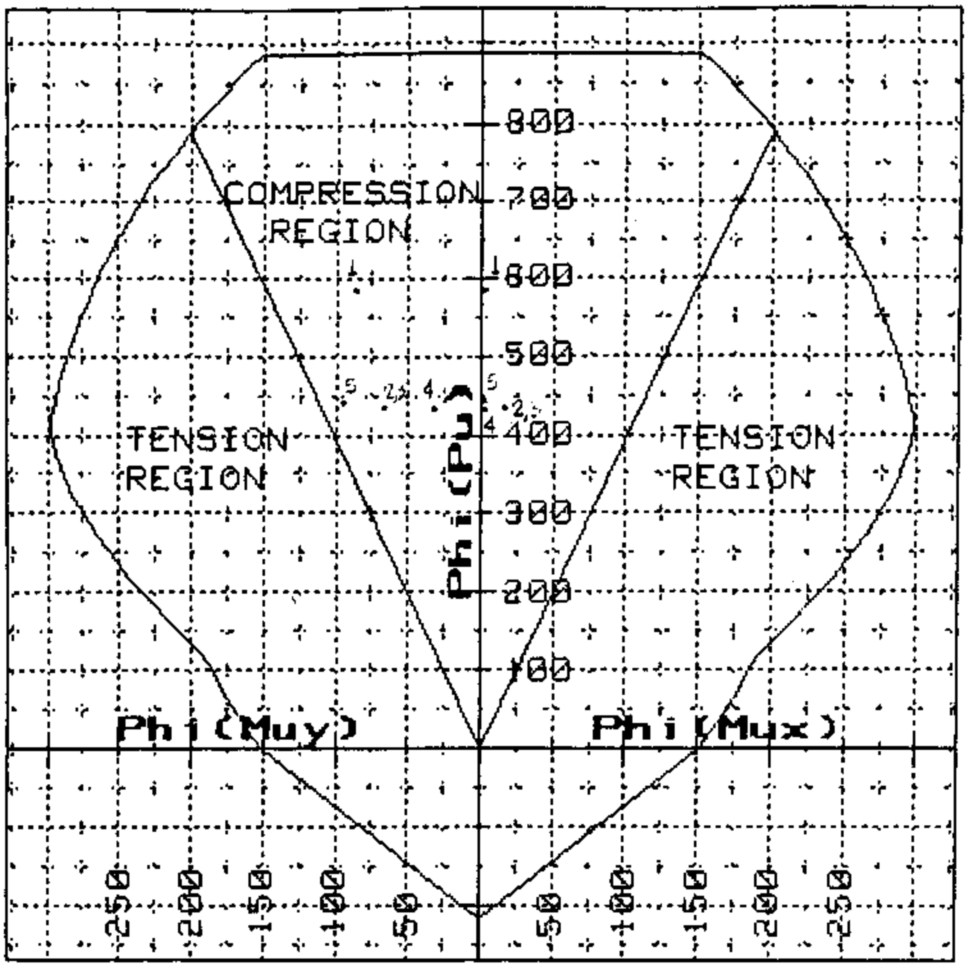
+ MULTIPLY THE LOAD CONDITION BY 0.710

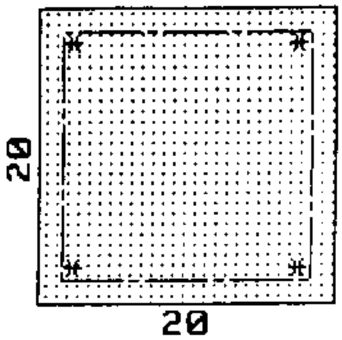
* MULTIPLY THE LOAD CONDITION BY 0.290

II COLUMN REINFORCEMENT SELECTION

THE CONCRETE COLUMN DESIGN AND REINFORCEMENT SELECTION WAS DONE USING CONCRETE COLUMN INTERACTION DIAGRAMS. THE MEMBER LOADS FOR EACH COLUMN AT EVERY STORY WERE INPUT INTO THE HP41CV COLUMN COMBINATION PROGRAM. (REFER TO APPENDIX #1 PGS 127-134). REFER TO PAGES 115 THRU 119 FOR THE SPECIFIC MEMBER LOAD COMBINATIONS. THE COLUMN COMBINATION PROGRAM CALCULATED AN AXIAL LOAD AND THE RESPECTIVE BIAXIAL BENDING MOMENTS, THESE POINTS WERE THEN PLOTTED ON THE DIAGRAMS. AN EXAMPLE OF THE CALCULATED POINTS AND THE USE OF THE DIAGRAMS IS ILLUSTRATED BELOW AND ON THE FOLLOWING PAGE.

```
COLUMN
A5-1 , D5-1
FRAME A,D
LC 1,2,5,6,9,10,11
FRAME 5
LC 1,2,5,7,8,9
EQ 1

P=584.454
M MAIN=0.000
M PERP=87.145

EQ 2

P=438.347
M MAIN=15.995
M PERP=65.359

P=438.347
M MAIN=15.995
M PERP=65.359

P=431.069
M PERP=33.347
M MAIN=0.000

P=445.598
M PERP=96.873
M MAIN=0.000
```



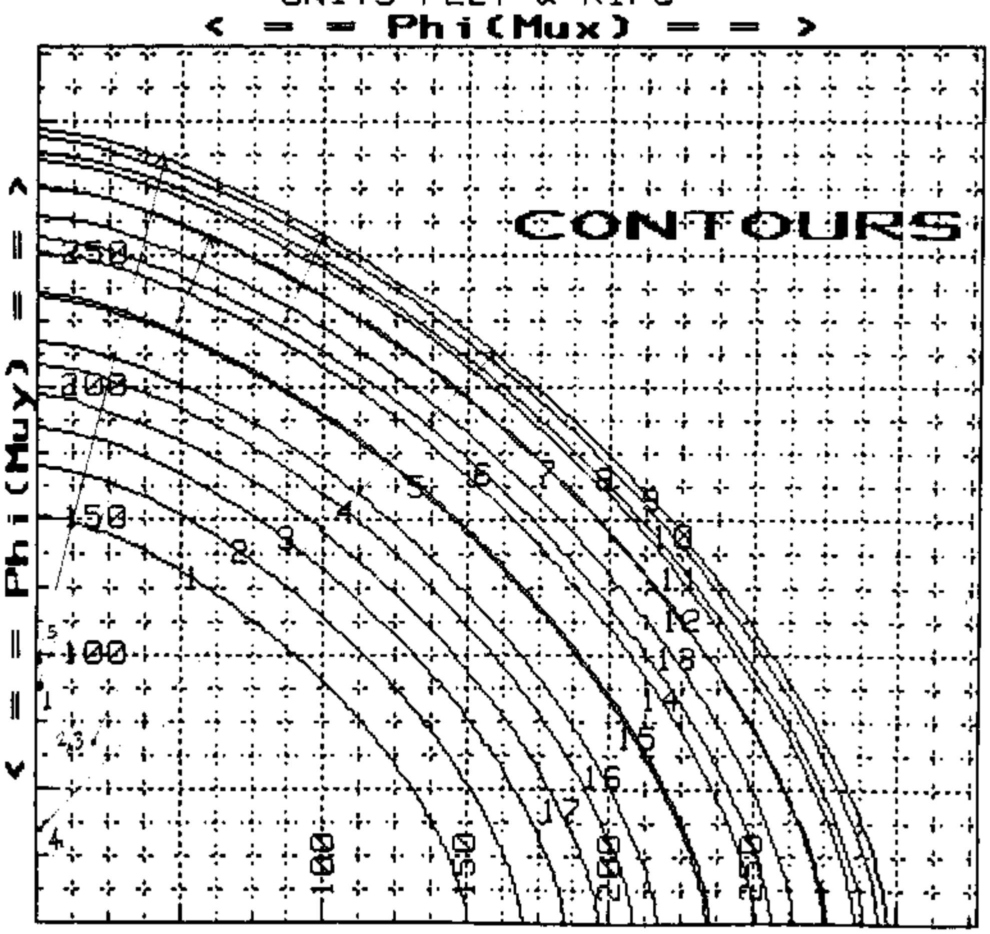


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

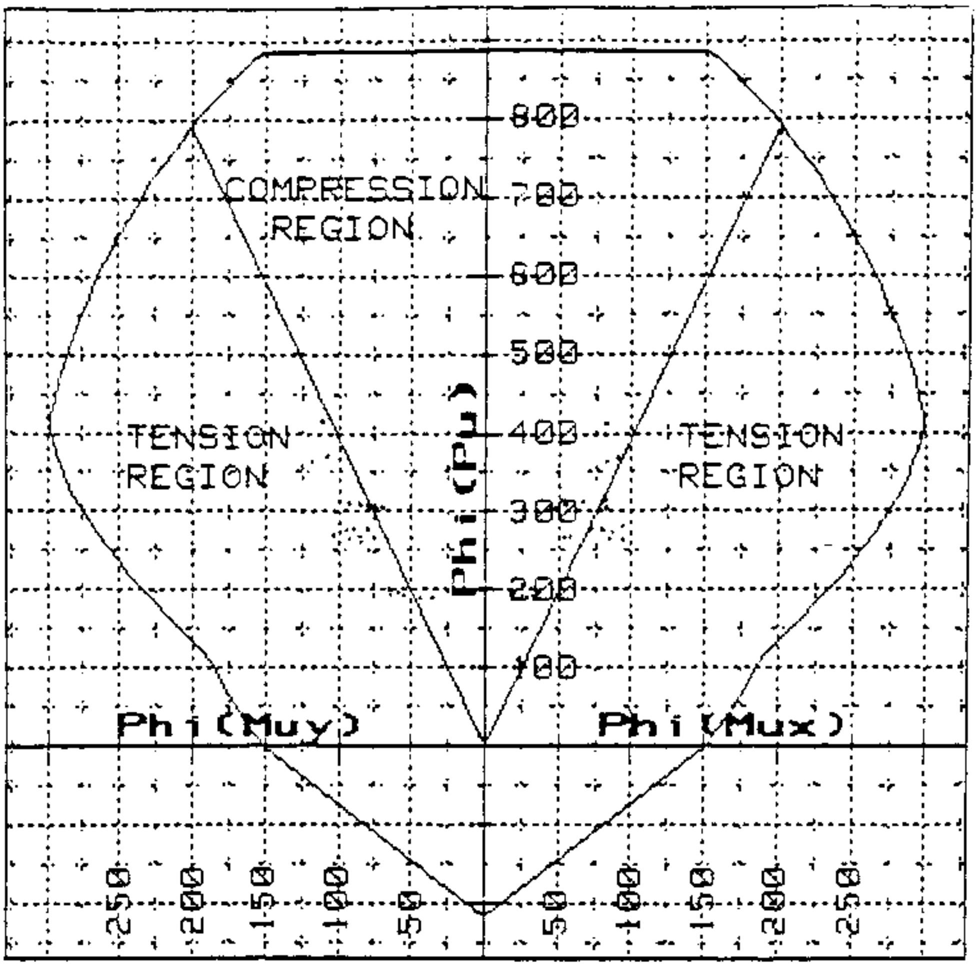
UNITS FEET & KIPS

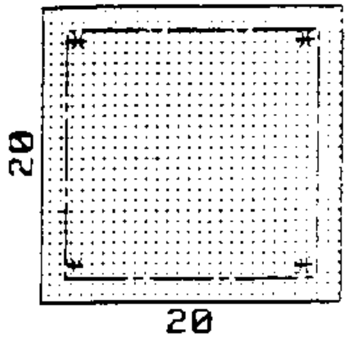


Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
3	100
4	150
5	200
6	250
7	300
8	350
9	400
10	450
11	500
12	550
13	600
14	650
15	700
16	750
17	800

EXAMPLE 121





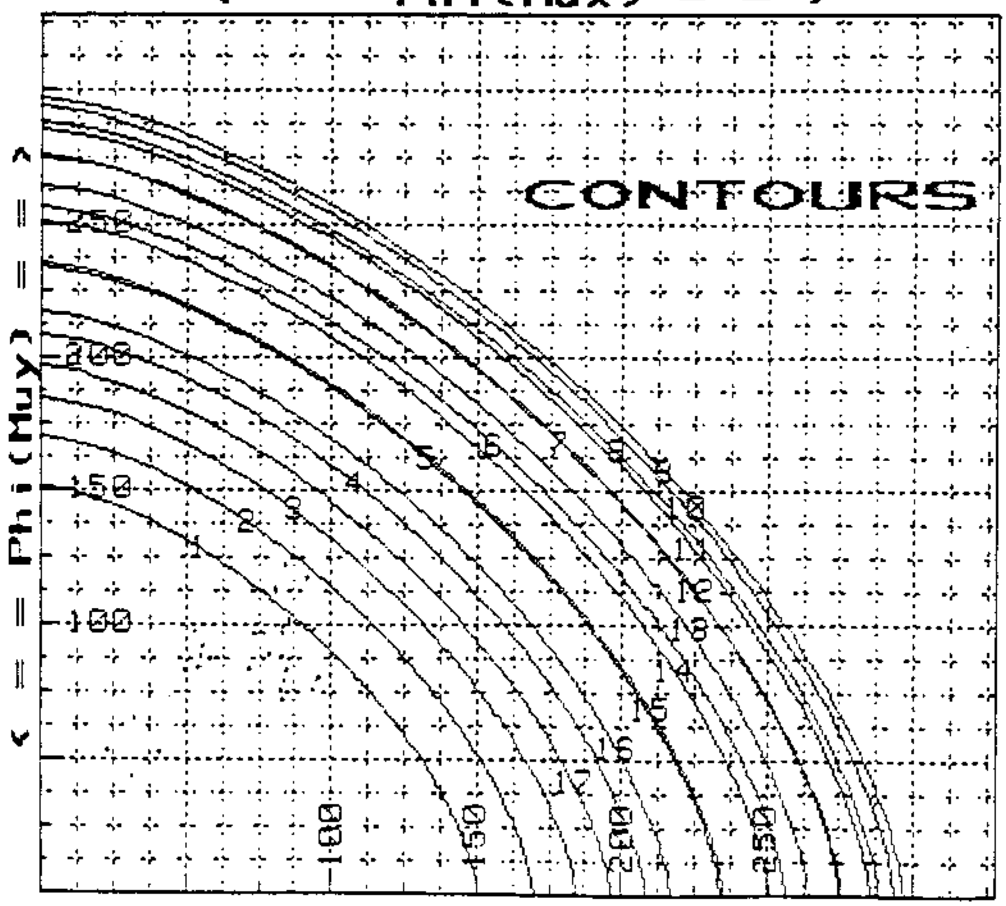
f'c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

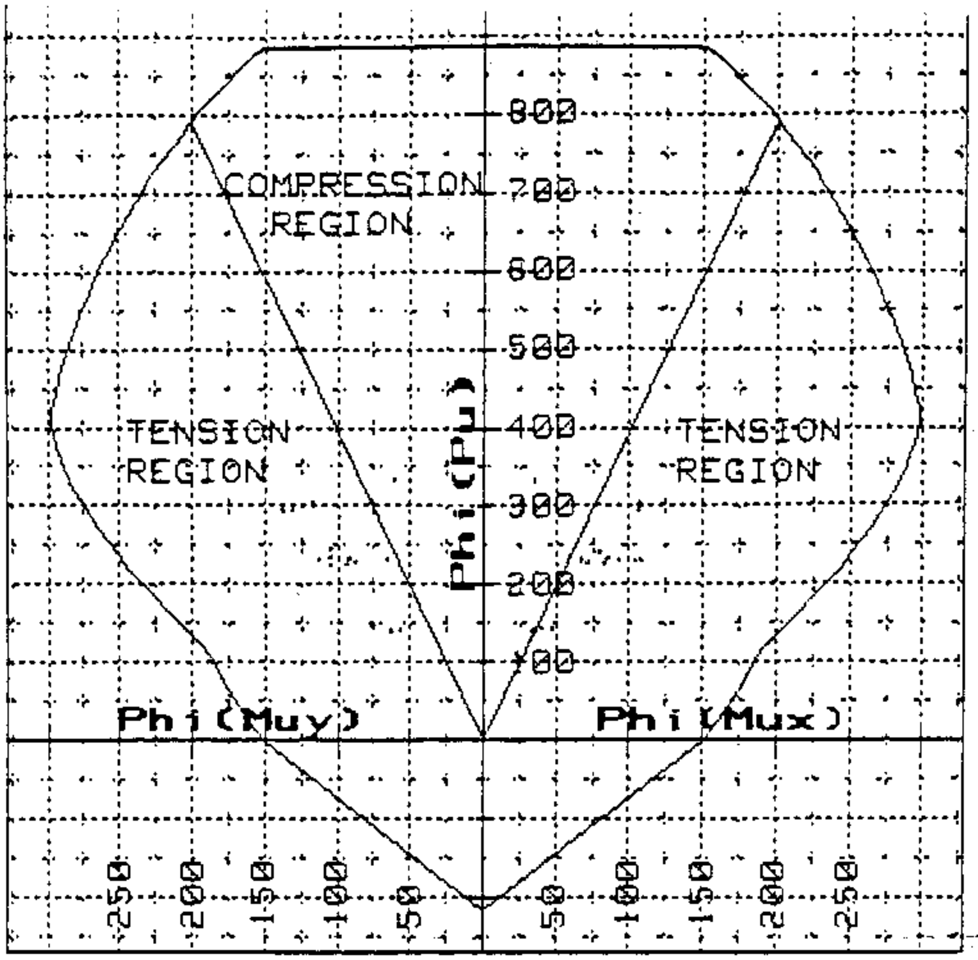
UNITS FEET & KIPS

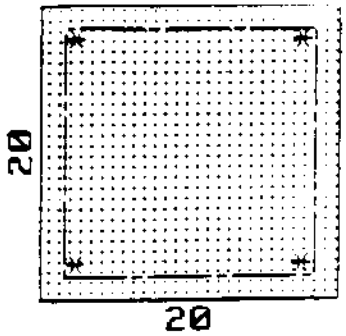
< = = Phi(Mux) = = >



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
3	100
4	150
5	200
6	250
7	300
8	350
9	400
10	450
11	500
12	550
13	600
14	650
15	700
16	750
17	800



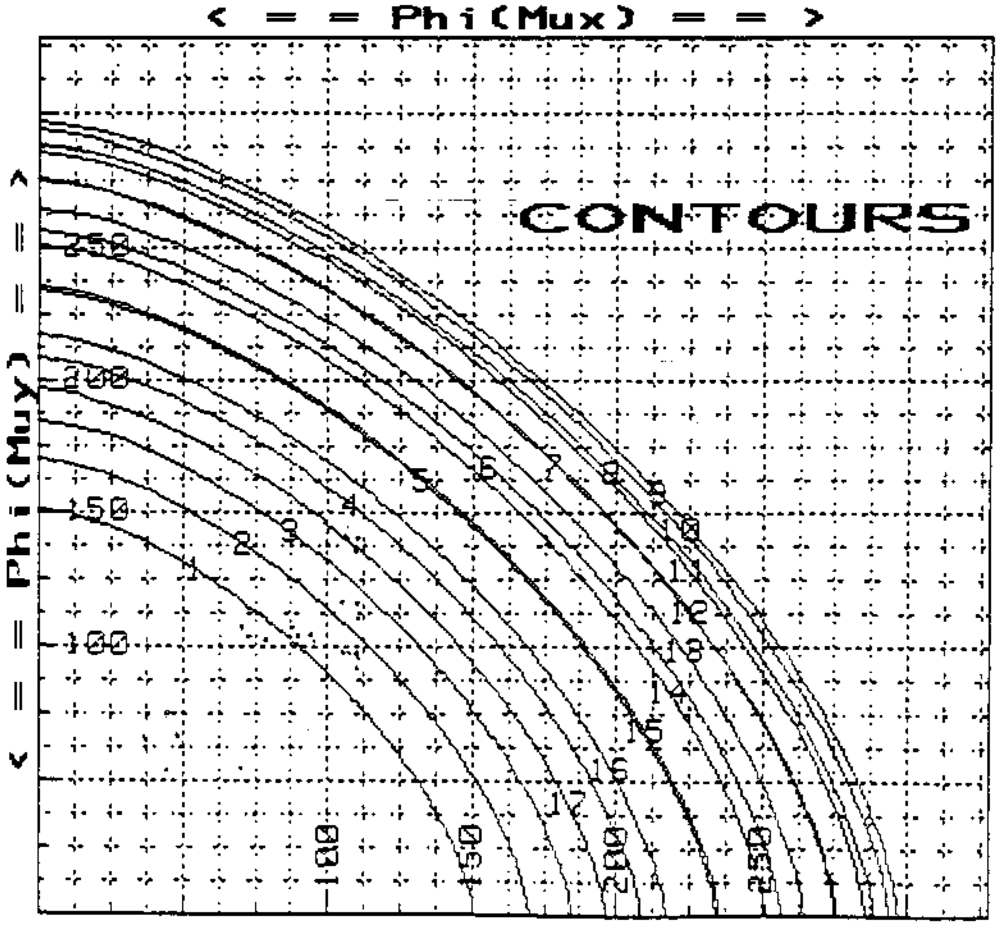


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

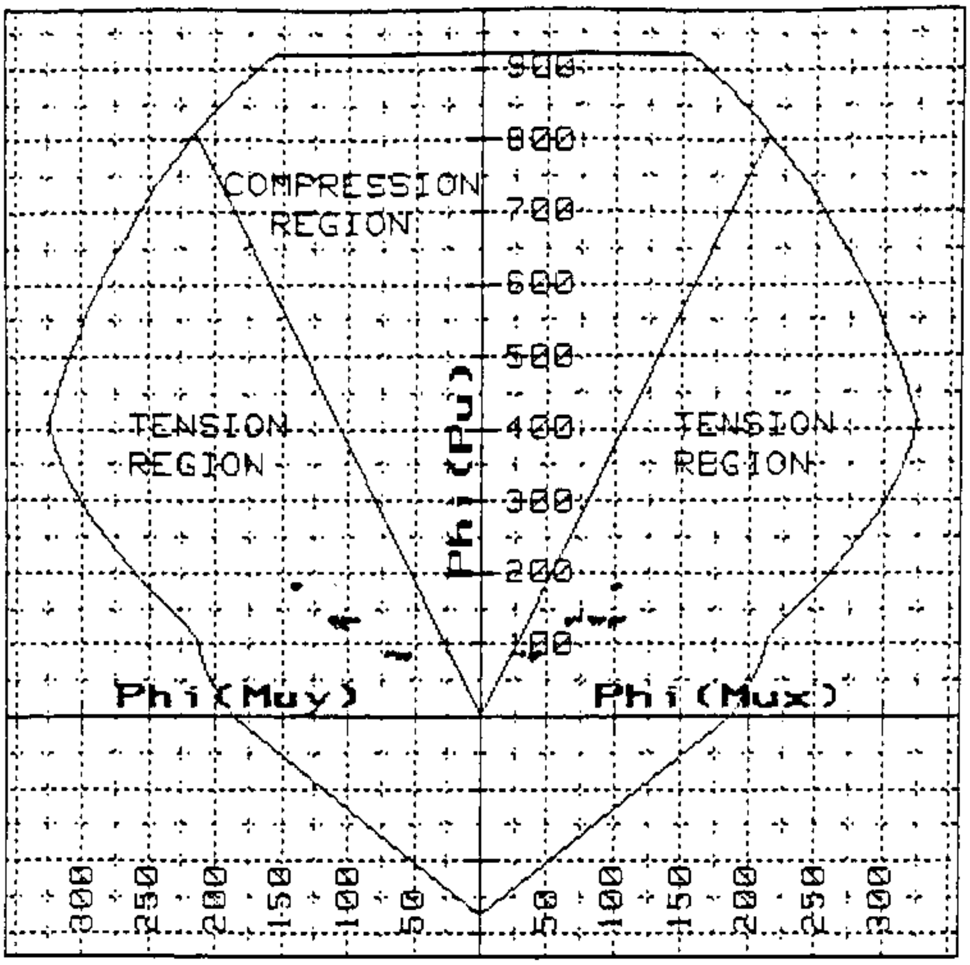
UNITS FEET & KIPS

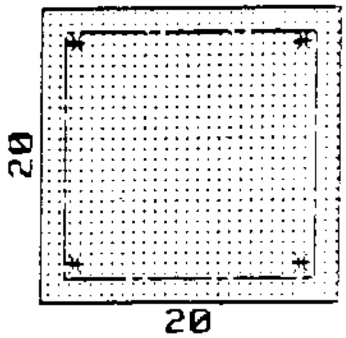


Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
3	100
4	150
5	200
6	250
7	300
8	350
9	400
10	450
11	500
12	550
13	600
14	650
15	700
16	750
17	800

AI-2, DI-2



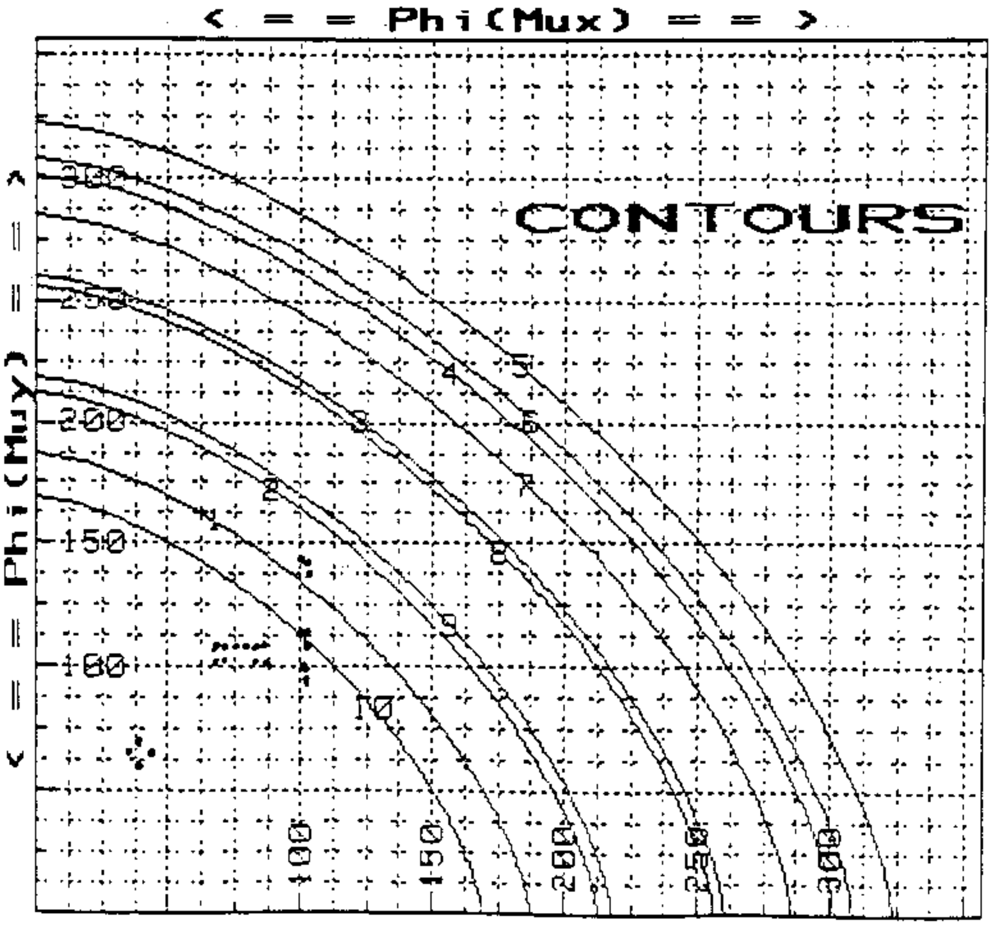


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 10
TOT. BARS= 4
Ast = 5.08
Rho = .0127
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

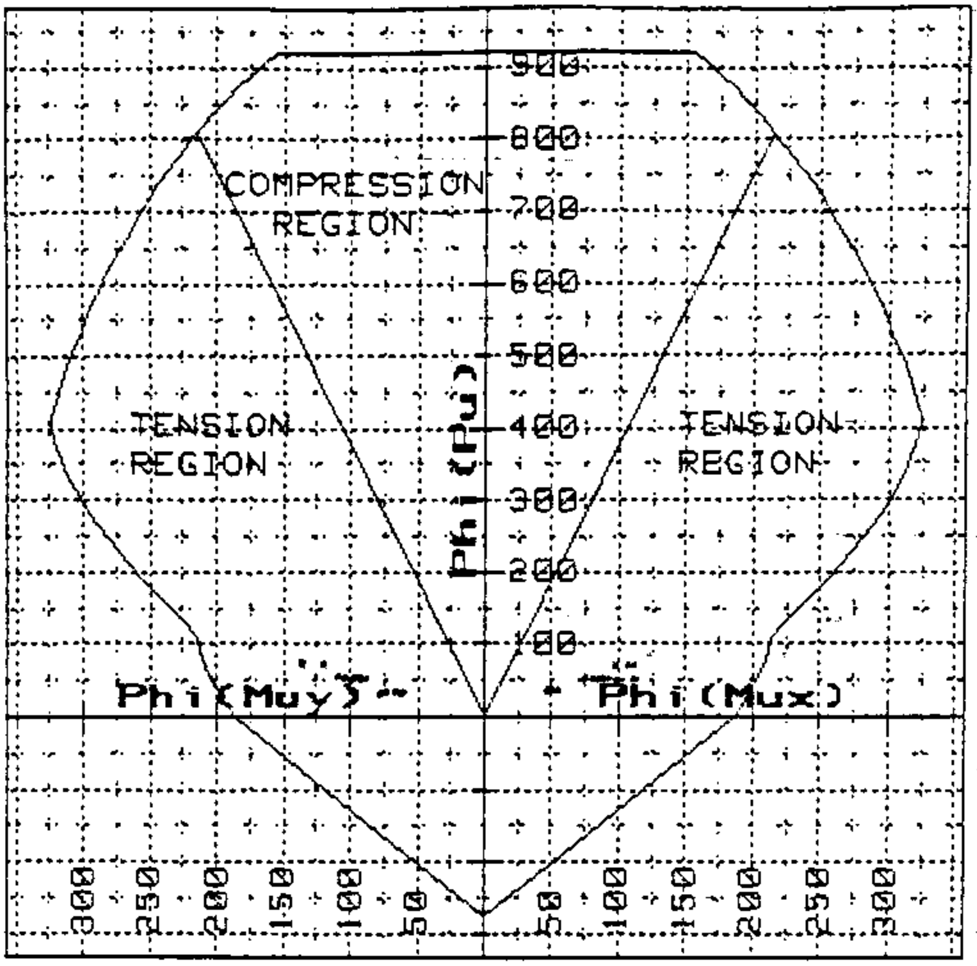
USE CLEAR COVER = 1.5 inches

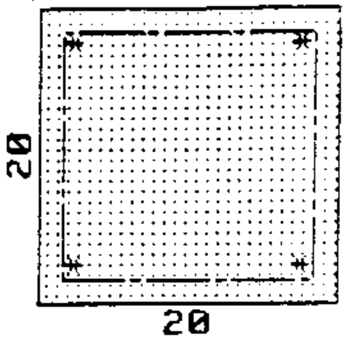
UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 284.98
Isey= 284.98

LINE	Pu
1	0
2	100
3	200
4	300
5	400
6	500
7	600
8	700
9	800
10	900



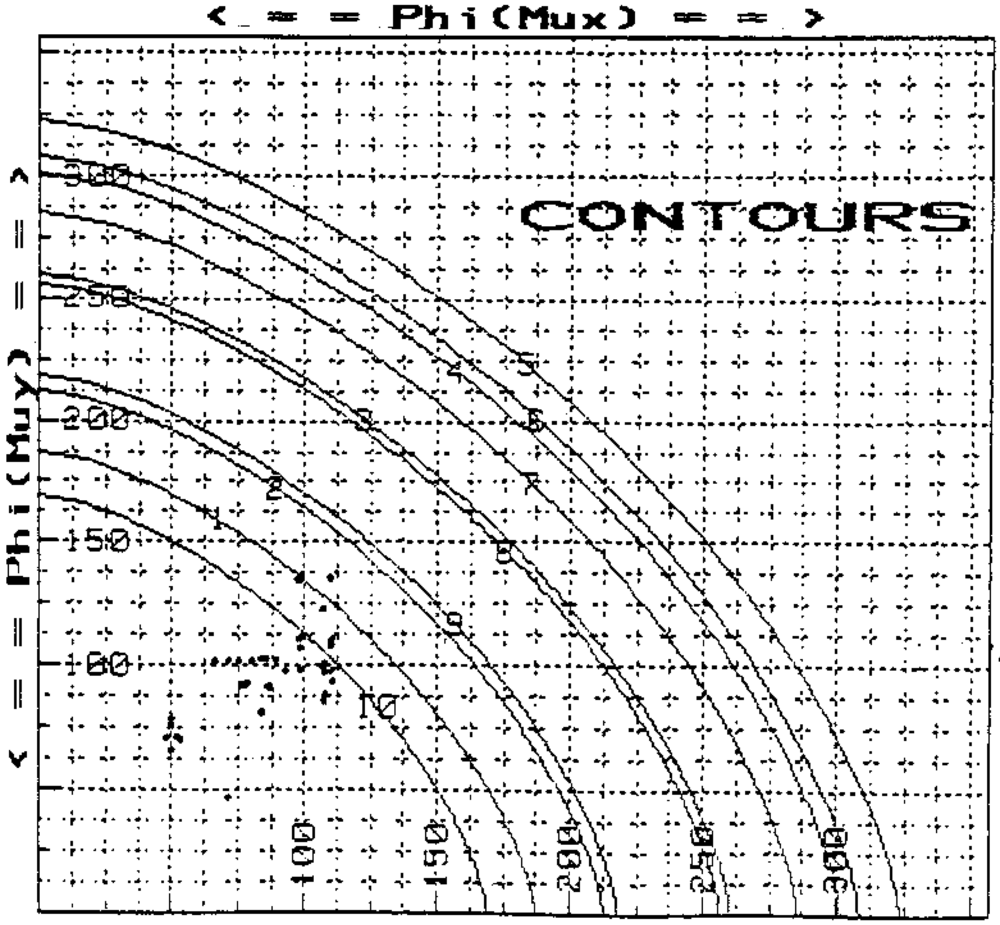


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 10
TOT.BARS= 4
Ast = 5.08
Rho = .0127
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 284.98
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LINE	Pu
1	0
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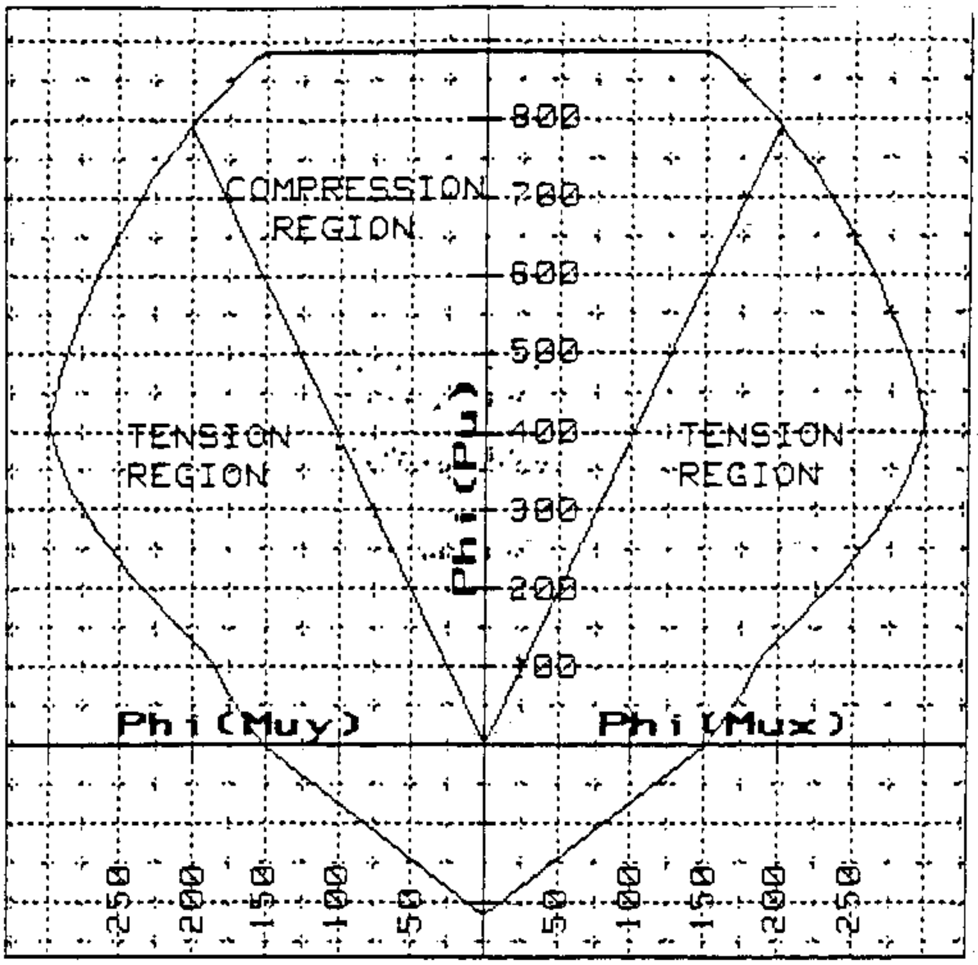
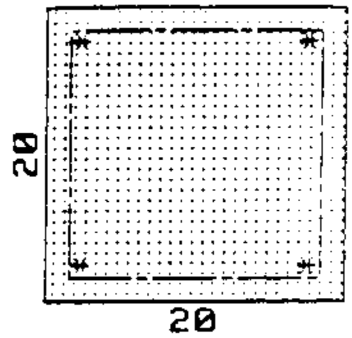
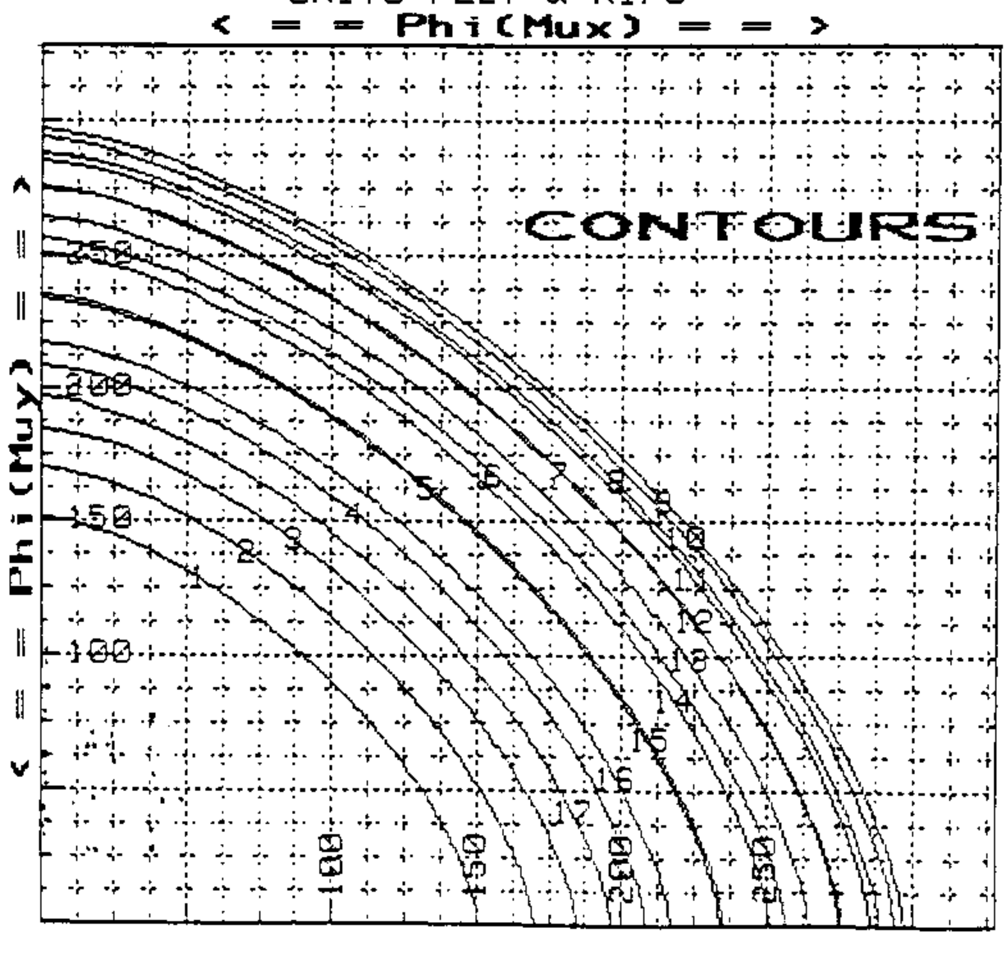




f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

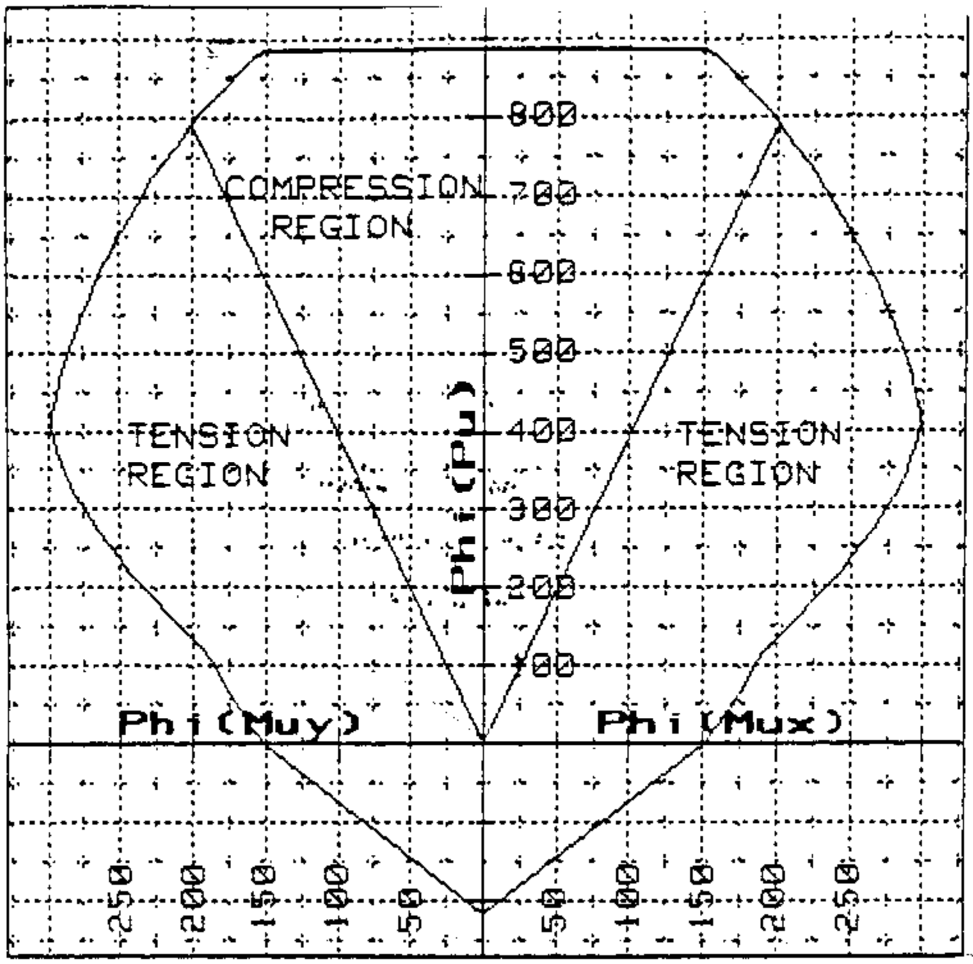
USE CLEAR COVER = 1.5 inches
UNITS FEET & KIPS

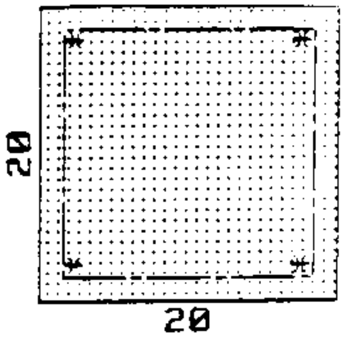


Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
3	100
4	150
5	200
6	250
7	300
8	350
9	400
10	450
11	500
12	550
13	600
14	650
15	700
16	750
17	800

A2-1, D2-1





f´c= 4 KSI
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# 3 TIES
@ 18 O.C.
CORNER BARS
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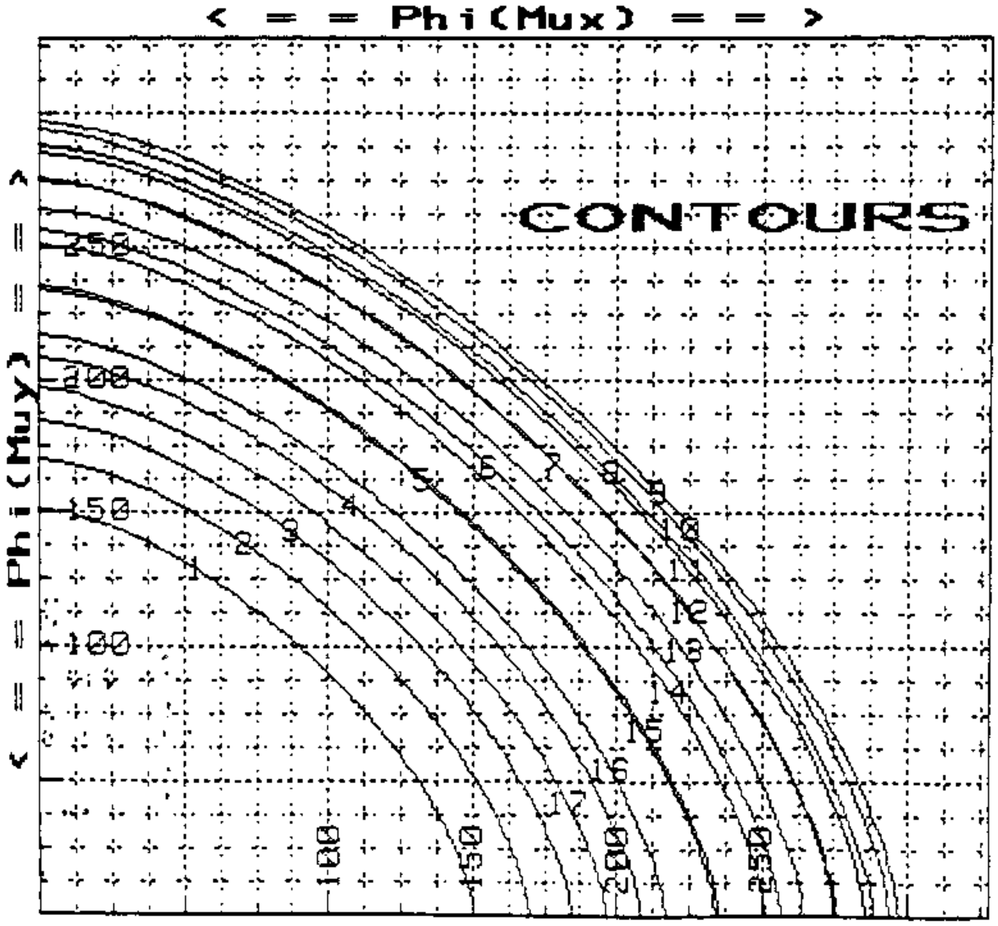
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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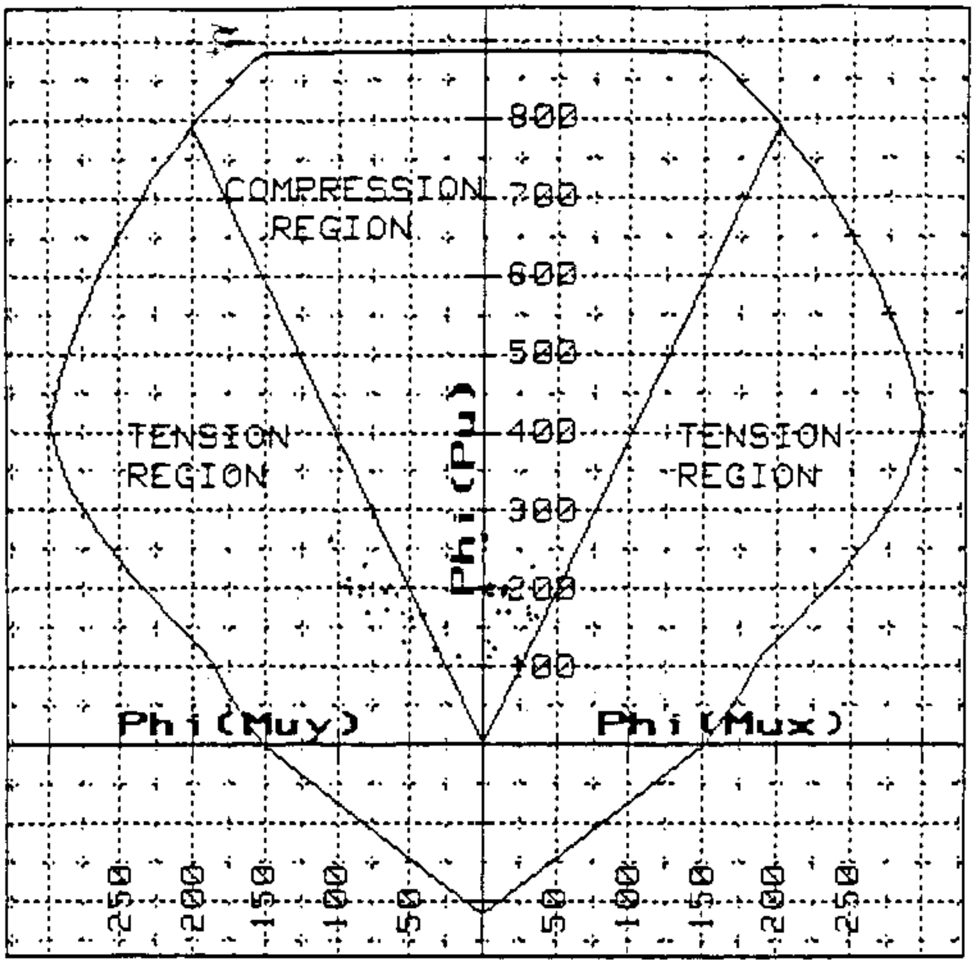

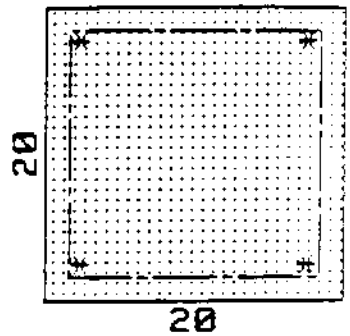

f´c= 4 KSI
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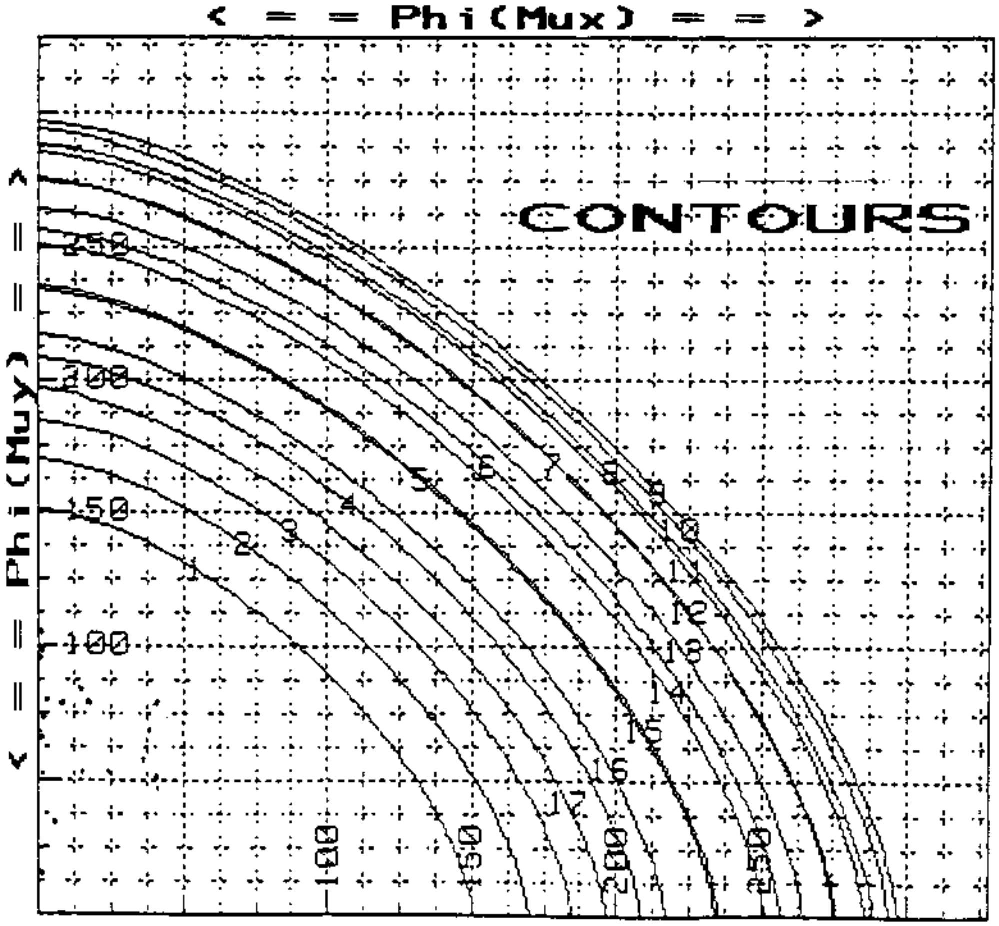
UNITS FEET & KIPS



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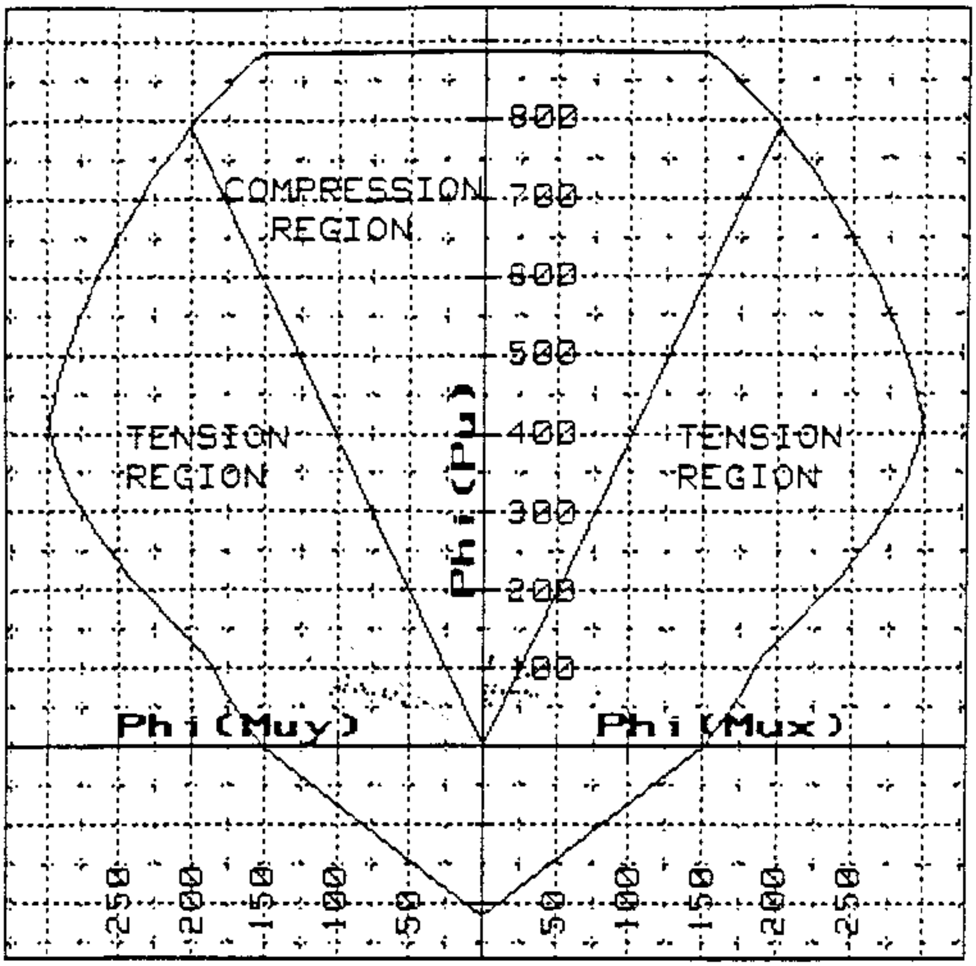

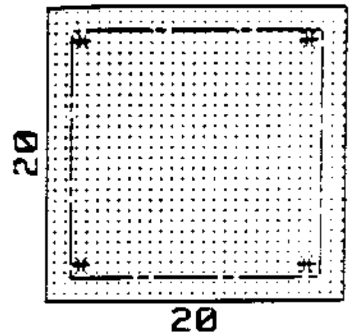

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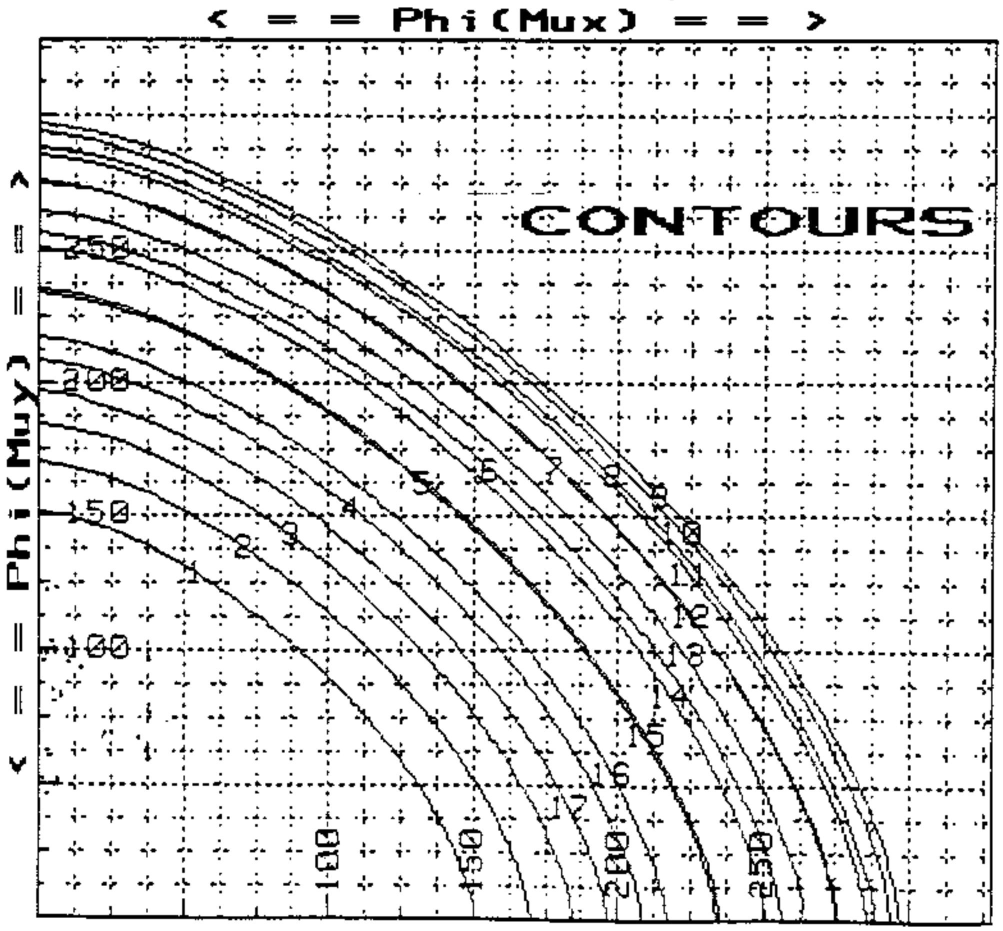
PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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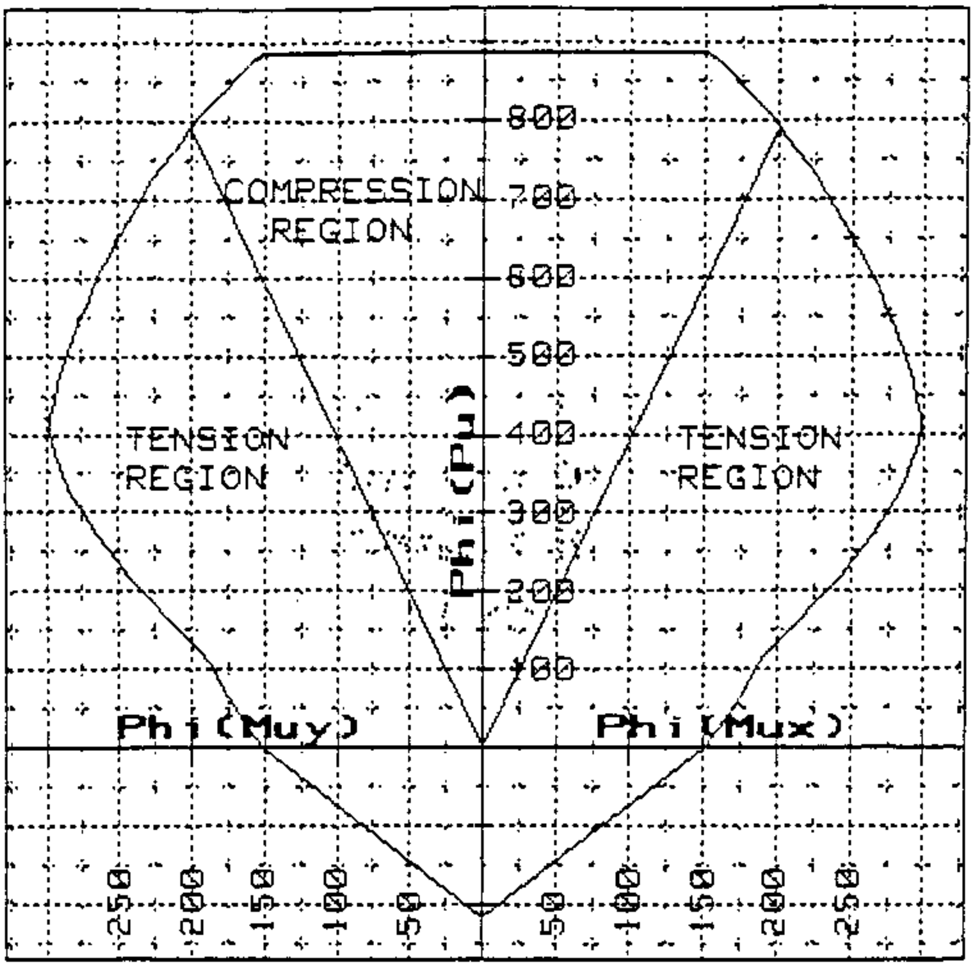

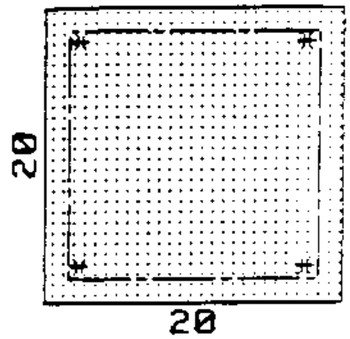

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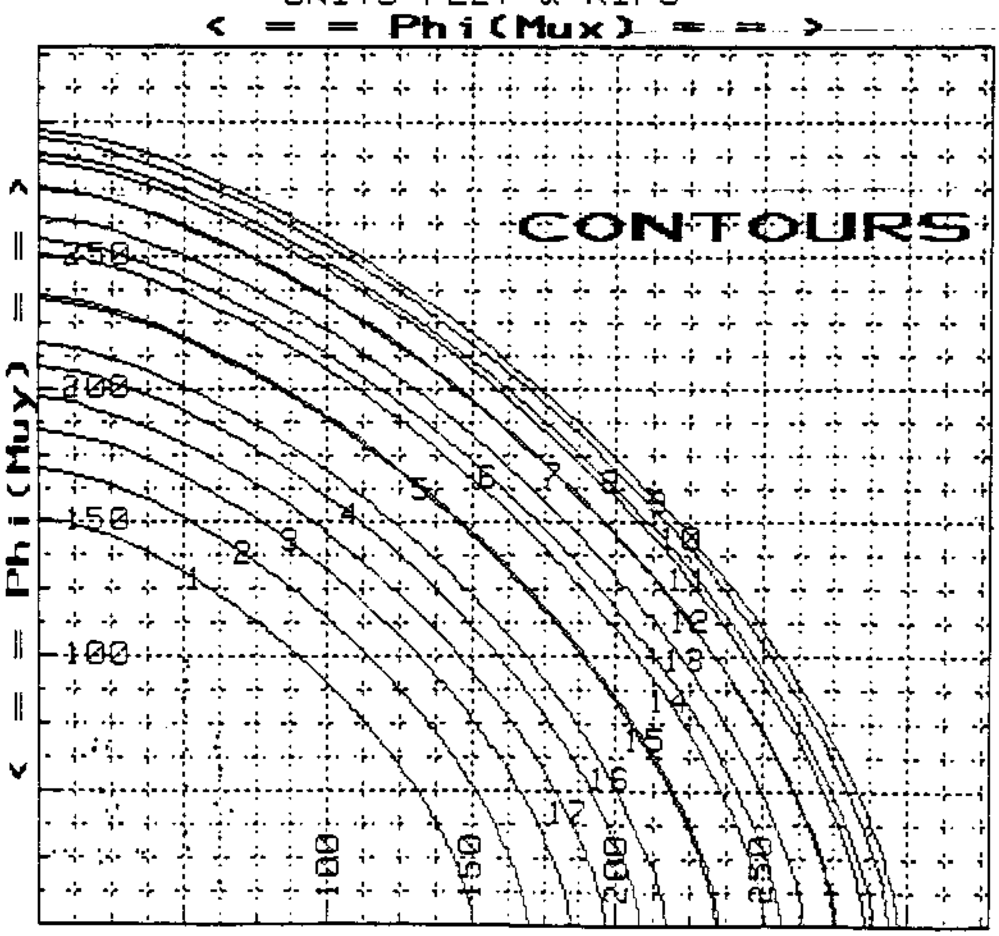
PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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11	500
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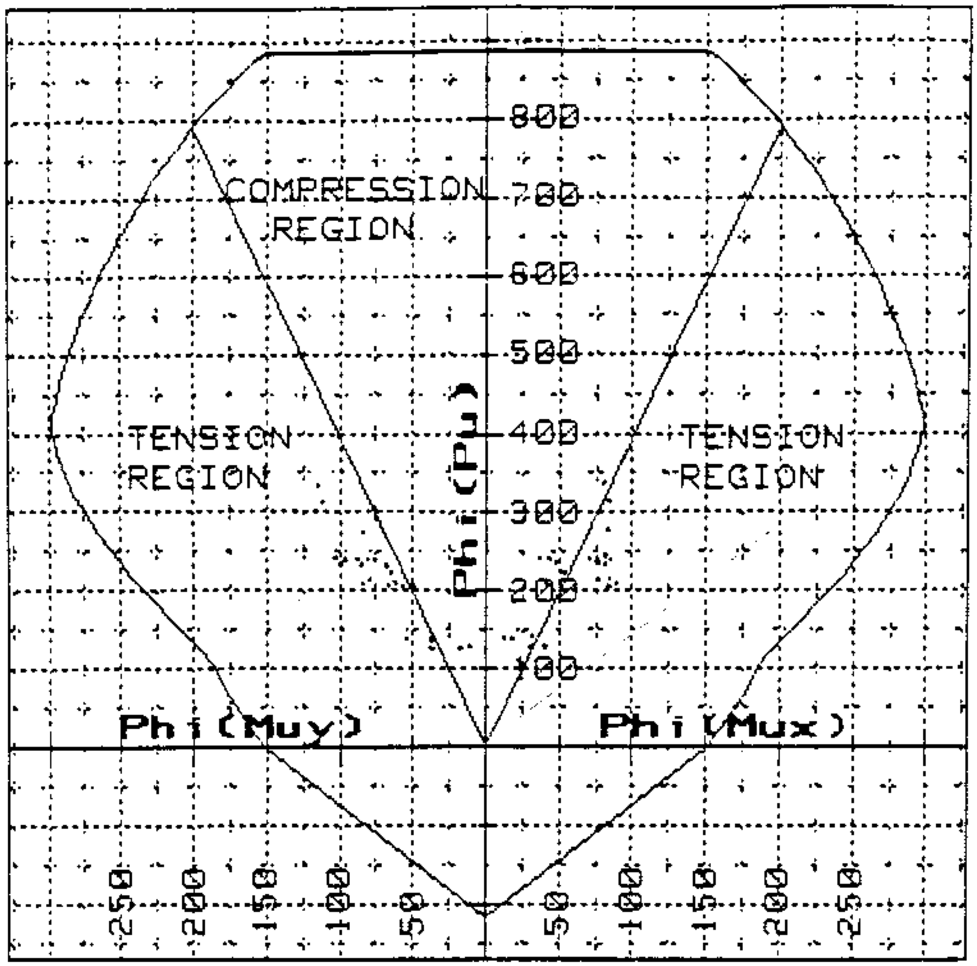
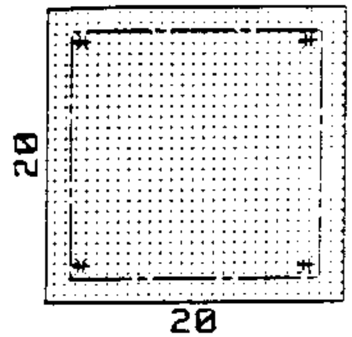
13	600
14	650
15	700
16	750
17	800

A7-1, D7-1
A7-1, D7-1





f´c= 4 KSI
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CORNER BARS
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TOT.BARS= 4
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Rho = .01
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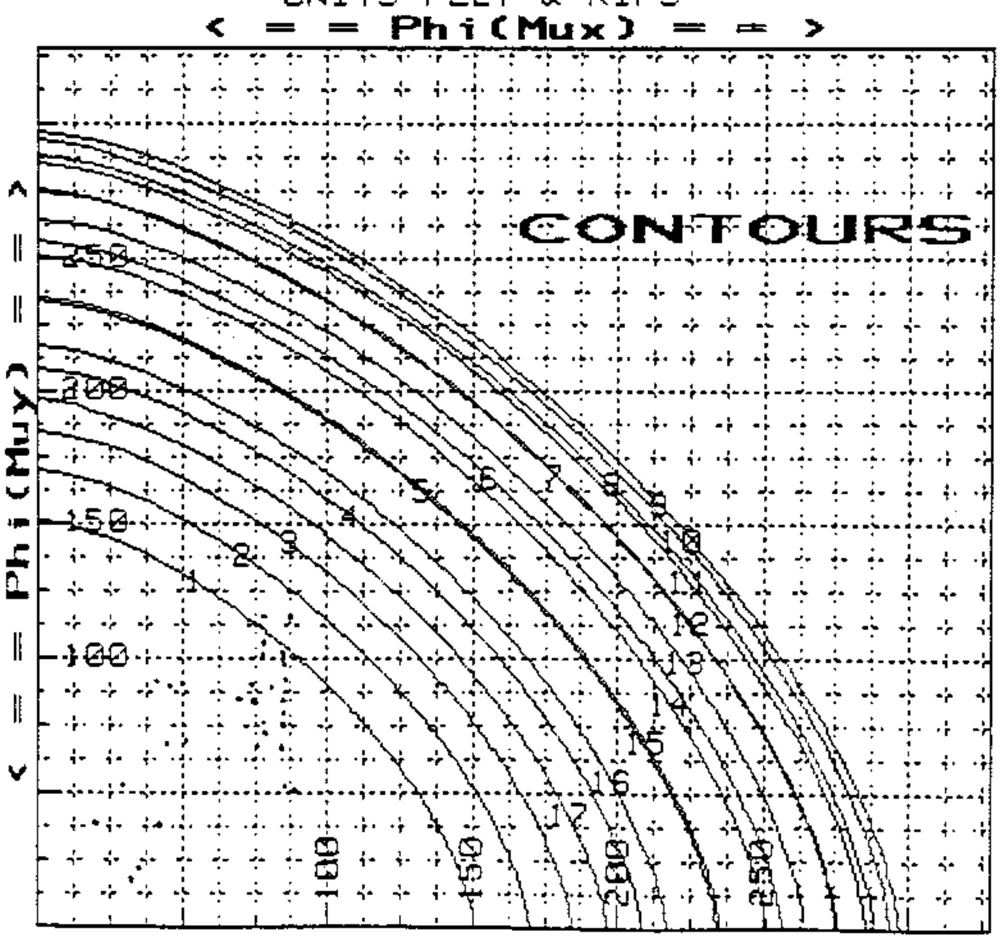
PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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11	500
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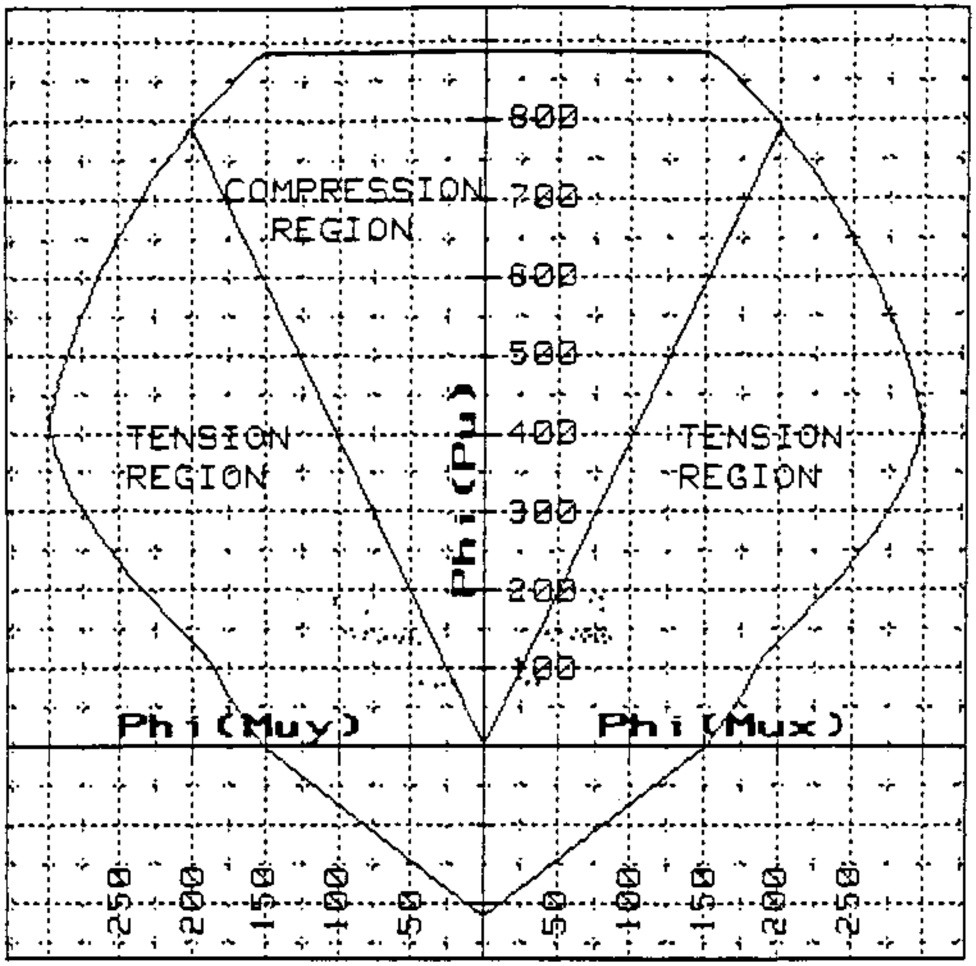
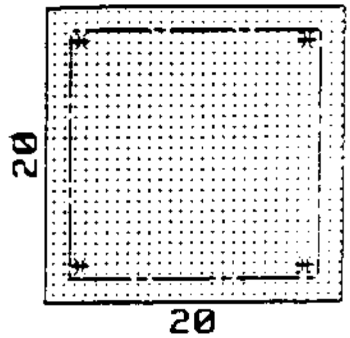
13	600
14	650
15	700
16	750
17	800

A7-2, D7-2
A8-2, D8-2





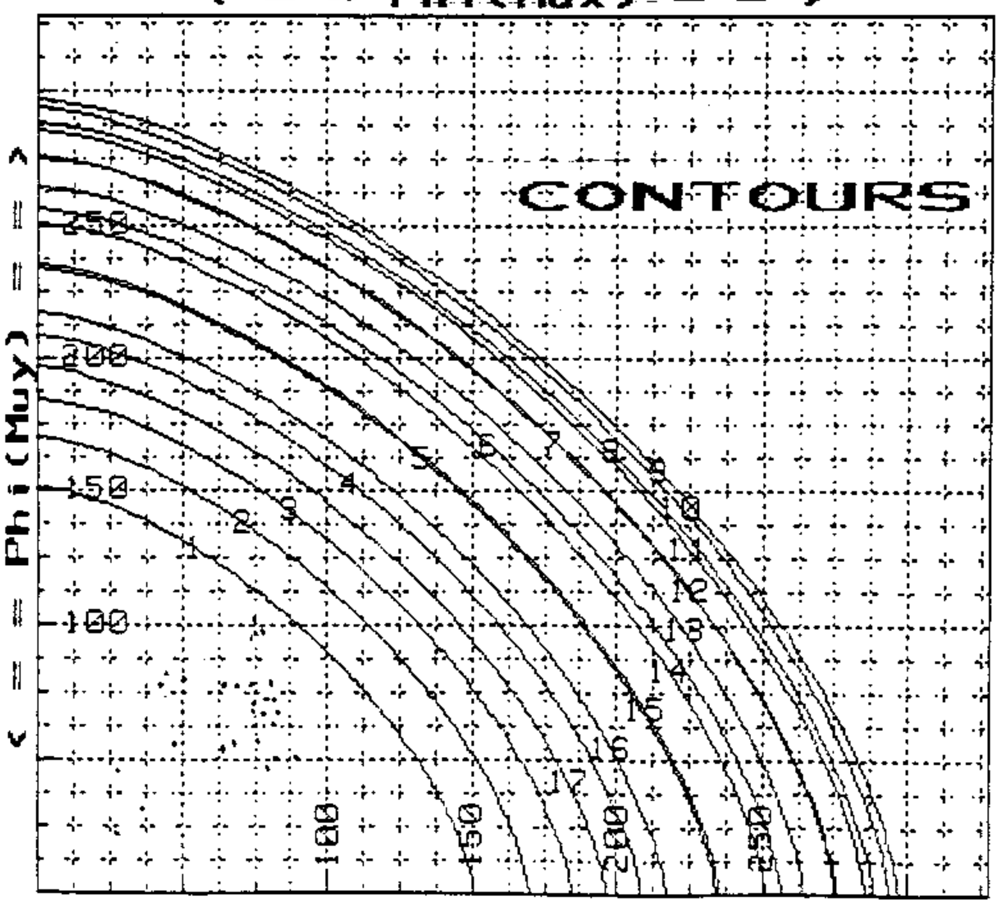
f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS

< = = Phi(Mux) = = >



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Ix, Iy Steel
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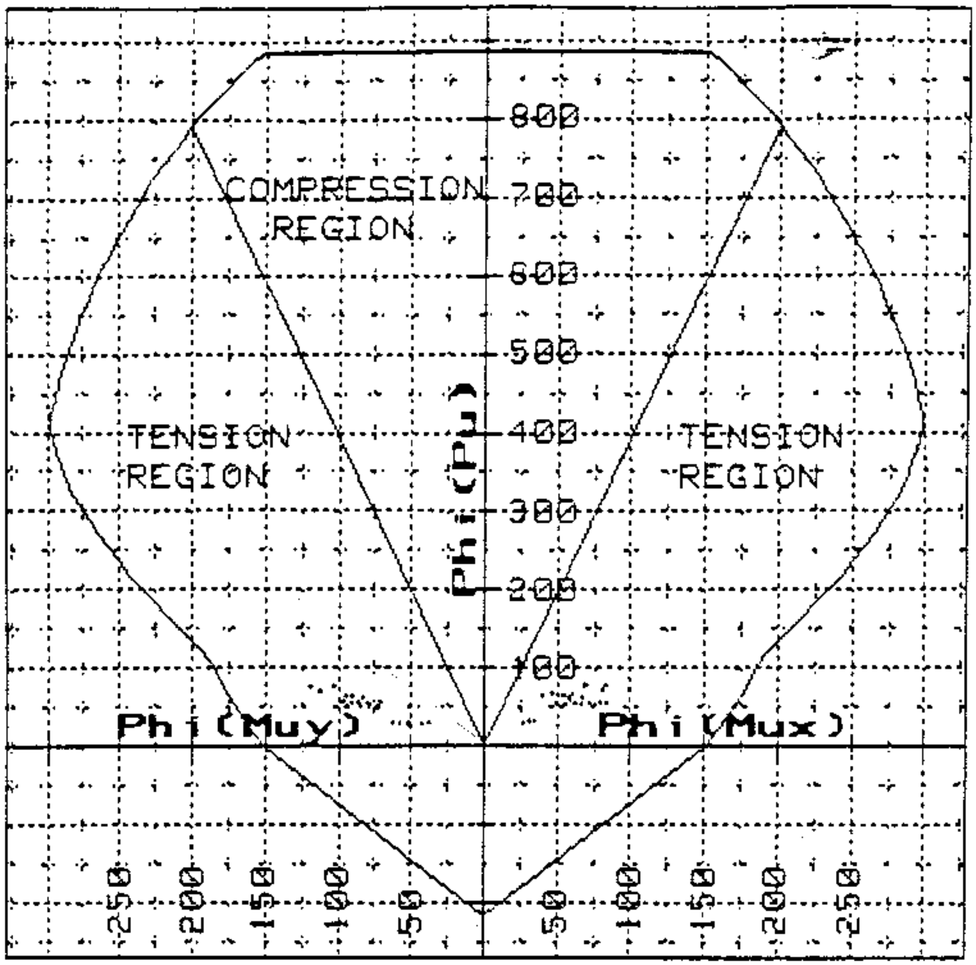
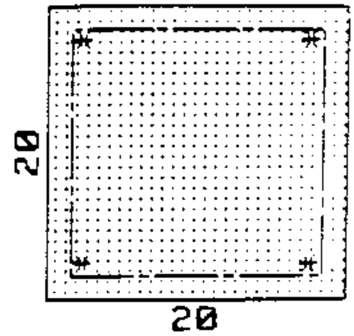
13	600
14	650
15	700
16	750
17	800

A7-3, D7-3
A3-3, D2-3



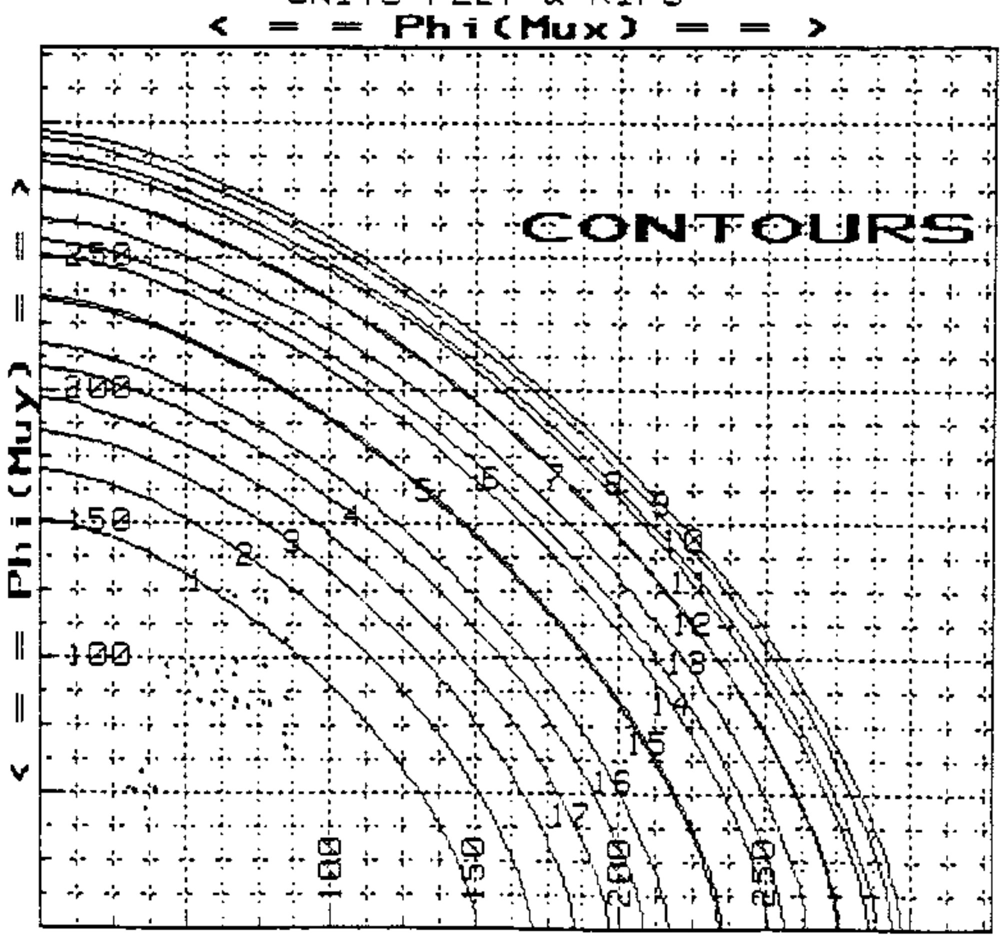


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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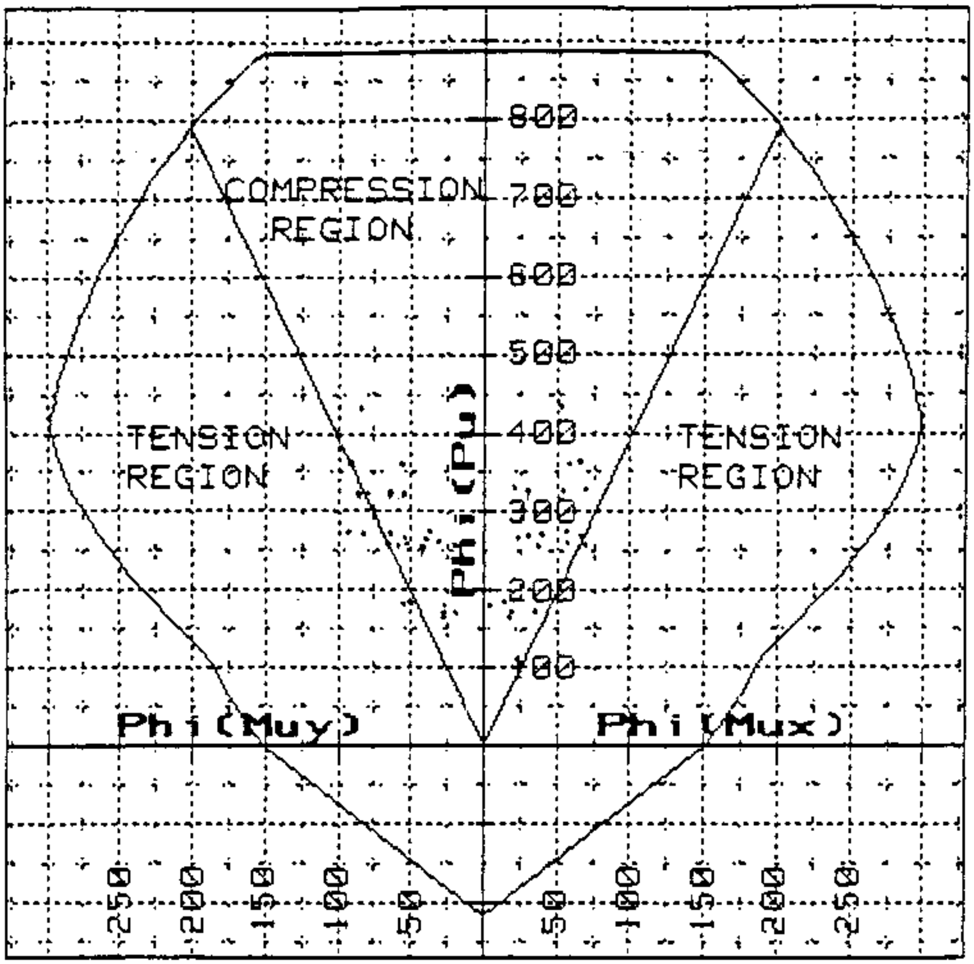
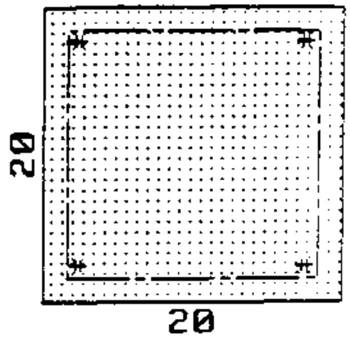
13	600
14	650
15	700
16	750
17	800

A7-4, D7-4
A8-4, D8-4



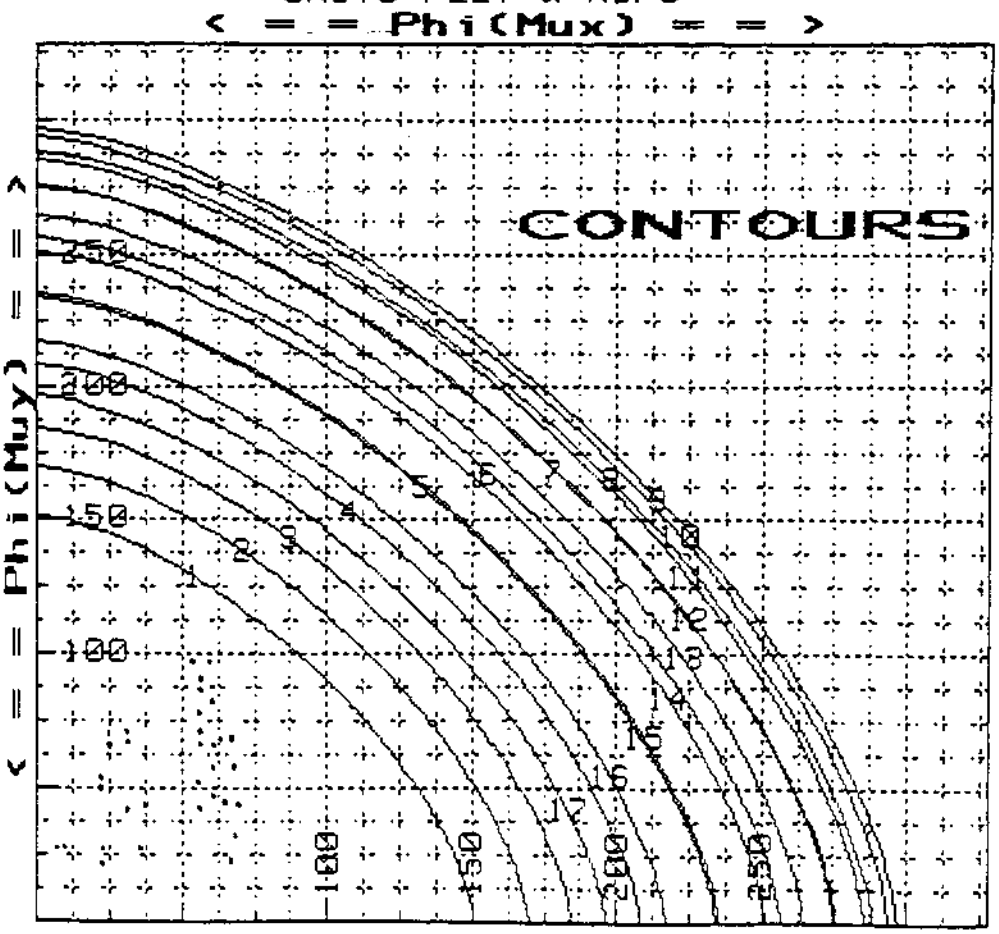


f'c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
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Ix, Iy Steel
Isex= 228.67
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LINE	Pu
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9	400
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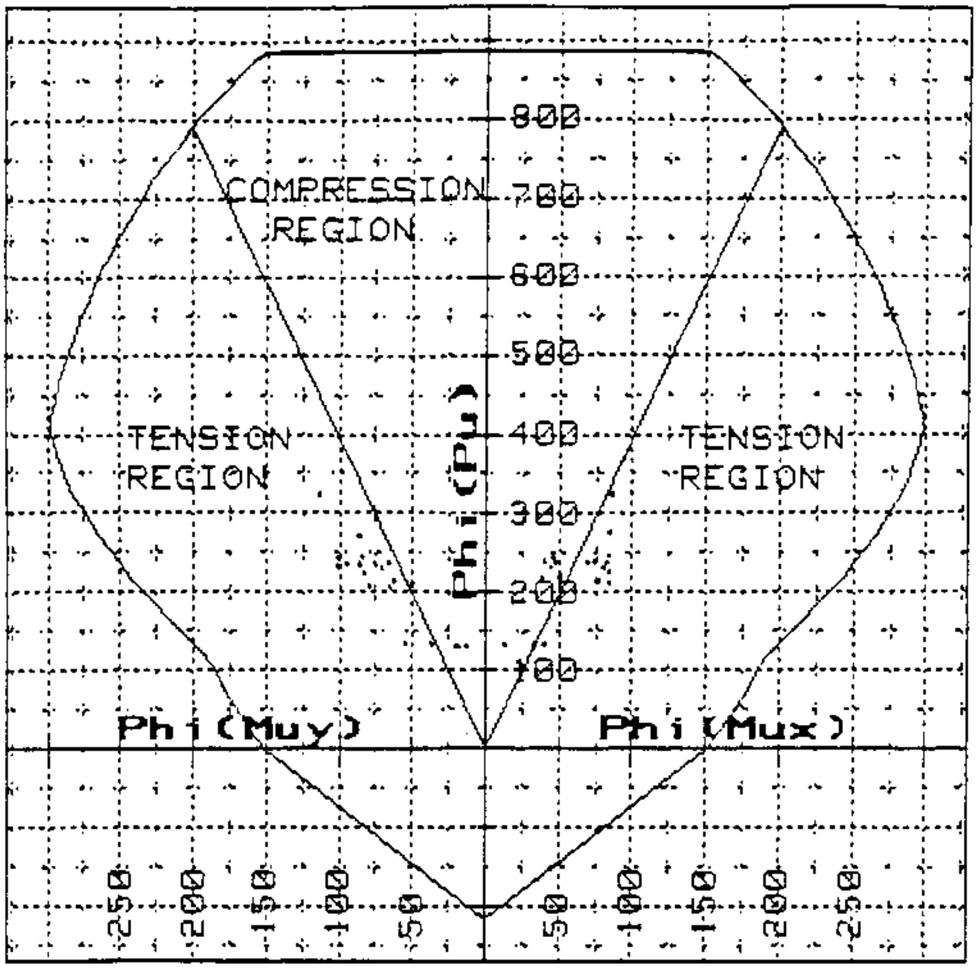
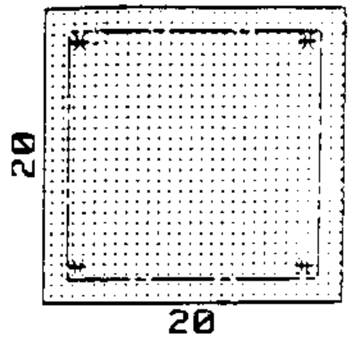
11	500
12	550
13	600
14	650
15	700
16	750
17	800

A6-1, D6-1
A4-1, D4-1



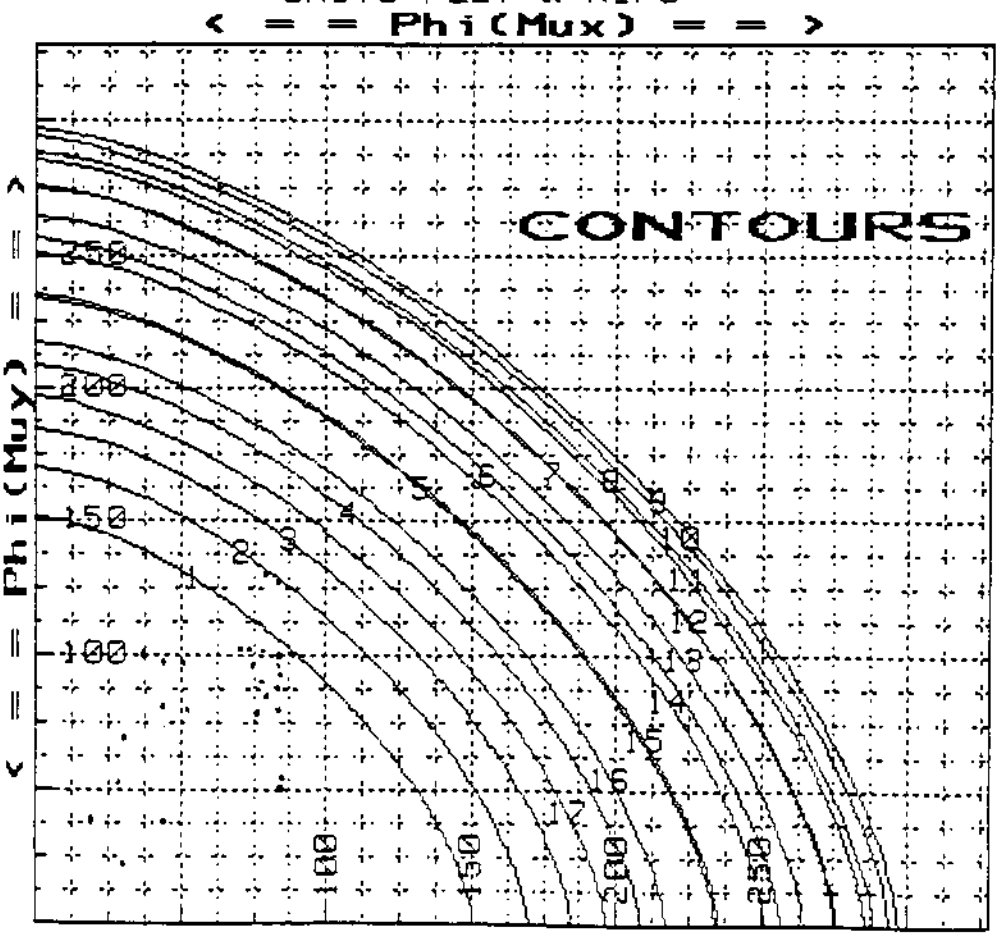


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
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LINE	Pu
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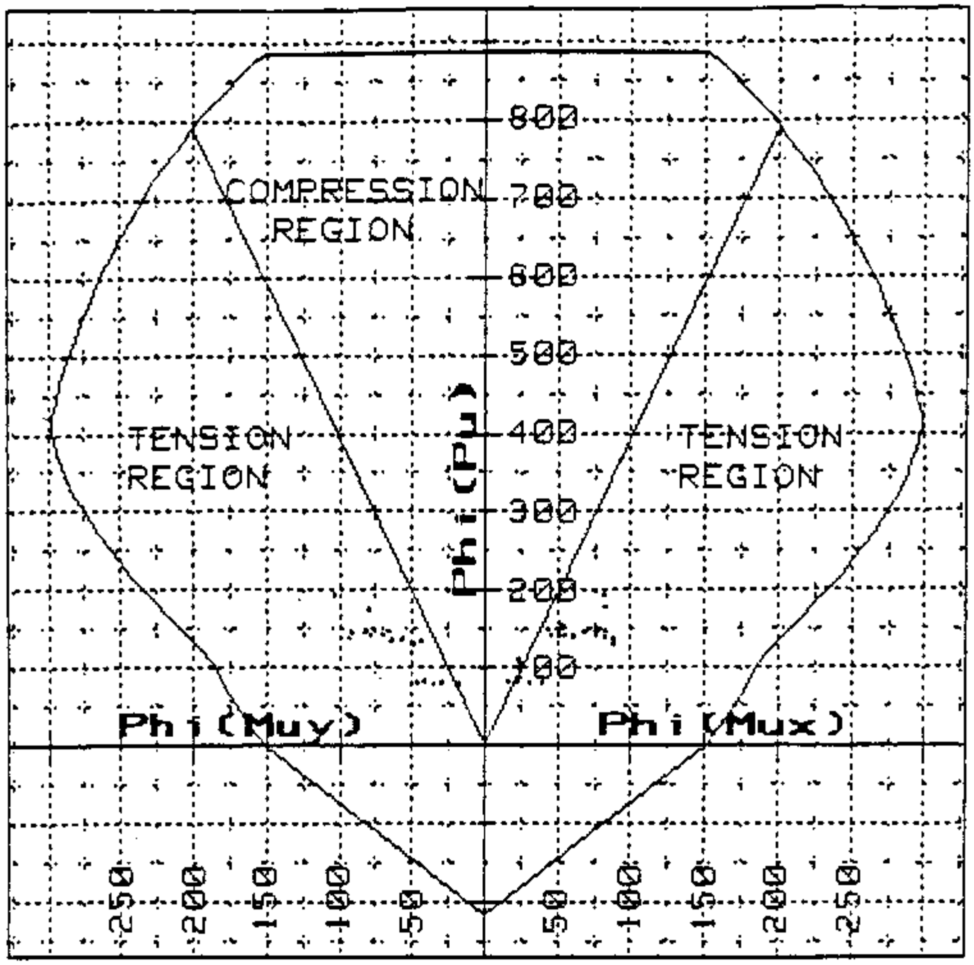
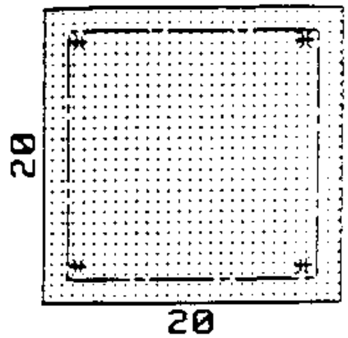
14	650
15	700
16	750
17	800

A6-2, D6-2
A4-2, D2-2





f´c= 4 KSI
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CORNER BARS
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PRINCIPAL AXES DIAGRAM

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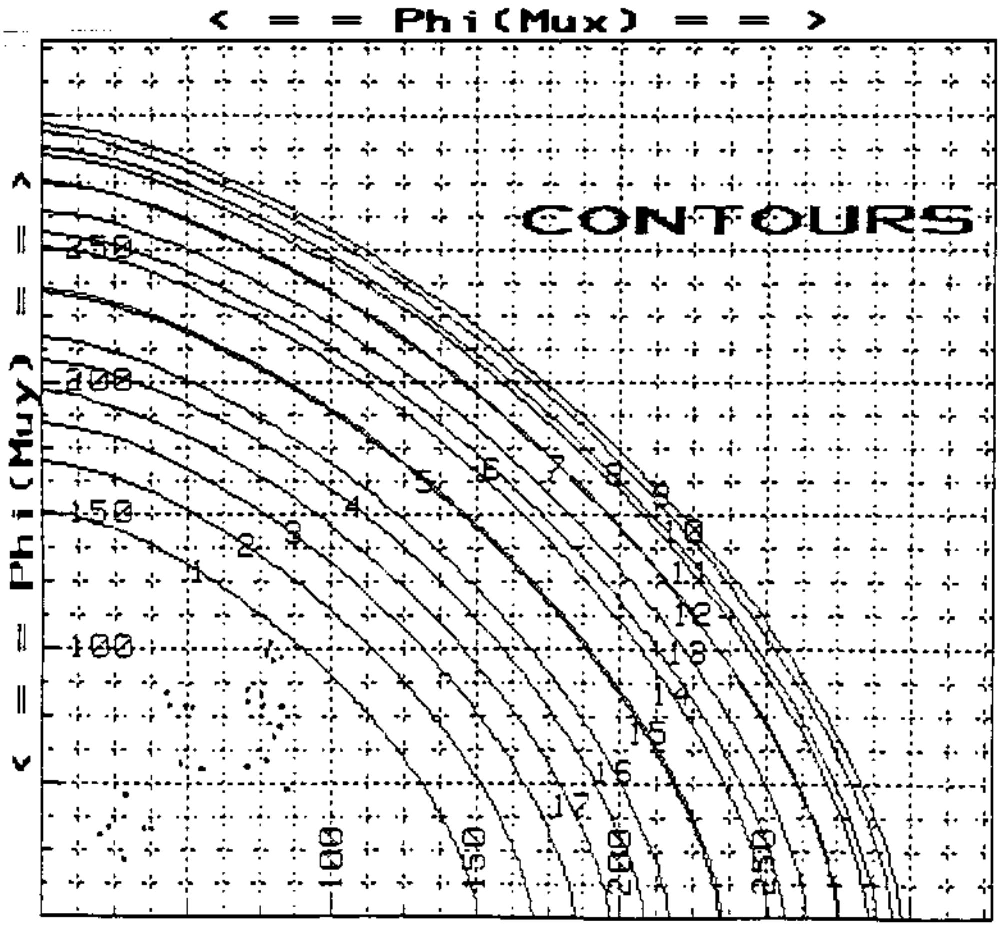
UNITS FEET & KIPS



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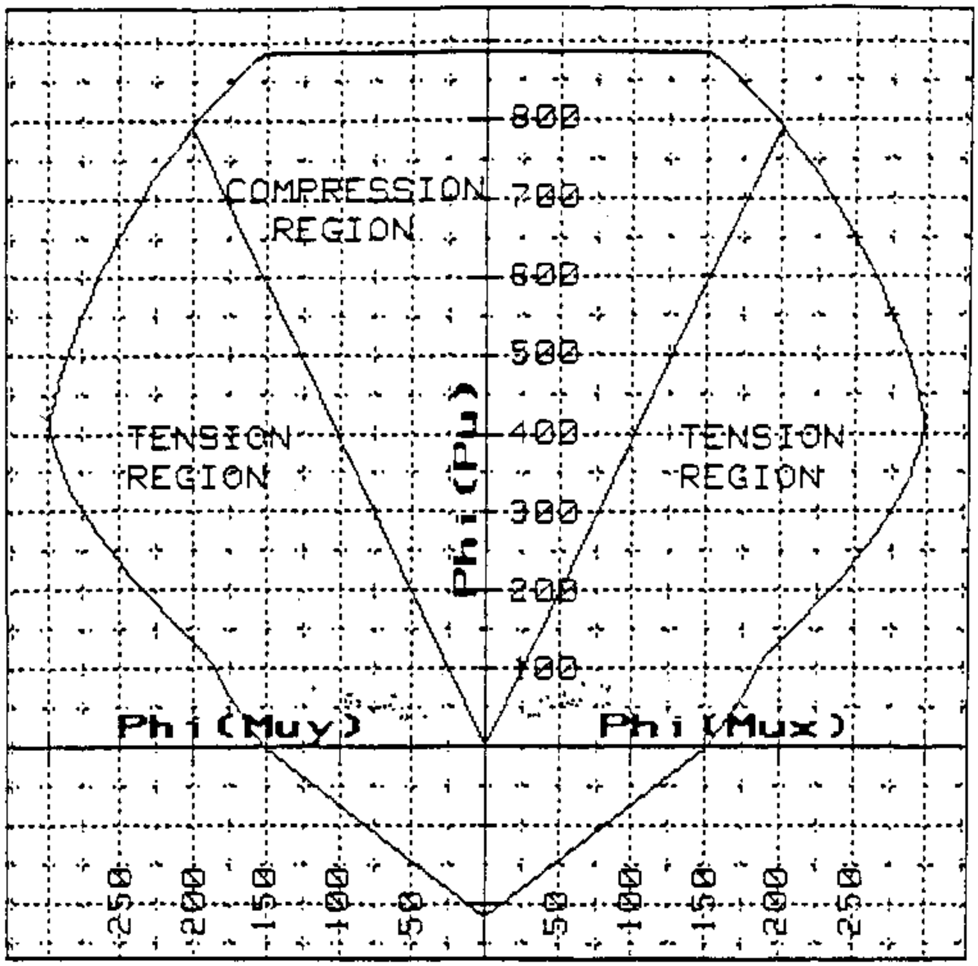
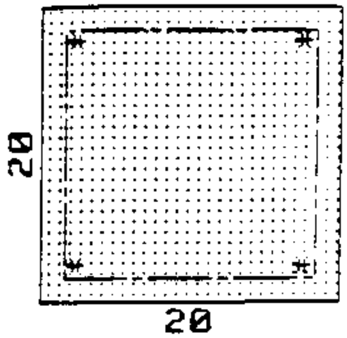
A6-3, D6-3
A1-3, D4-3





f´c= 4 KSI
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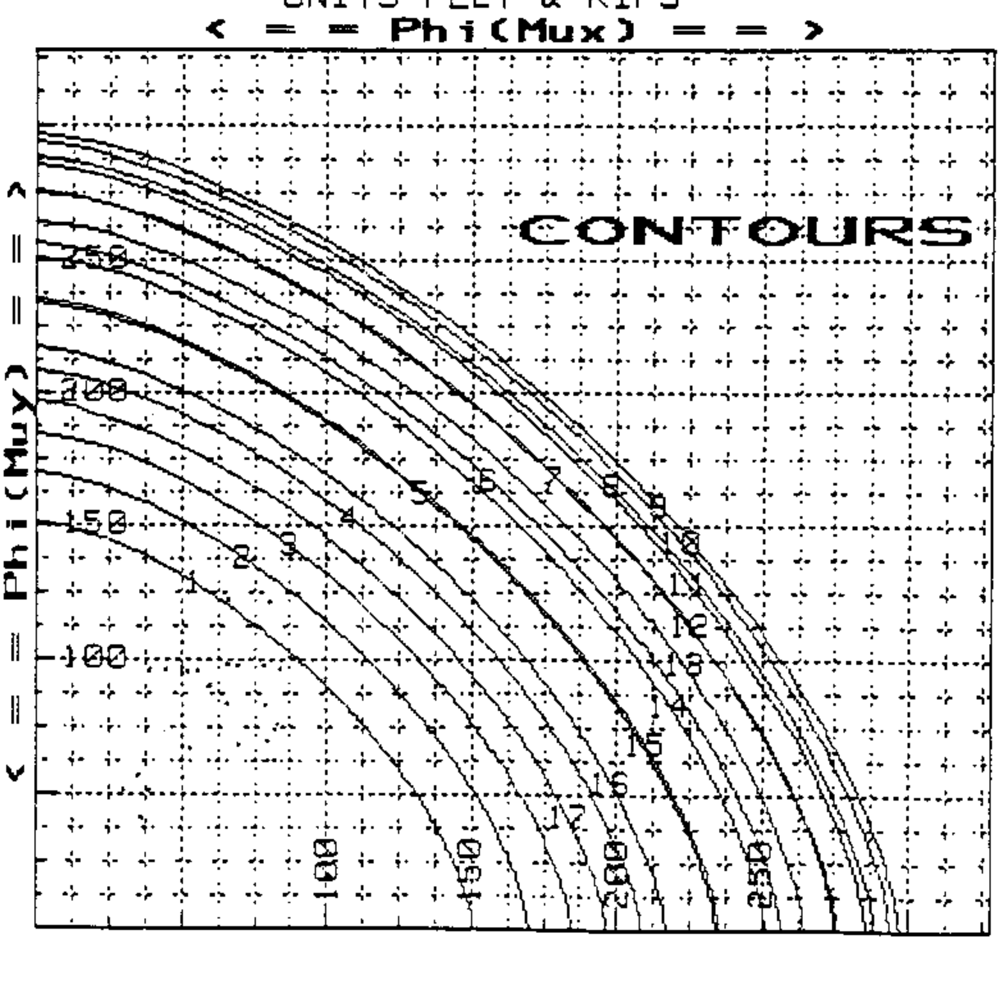
PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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15	700
16	750
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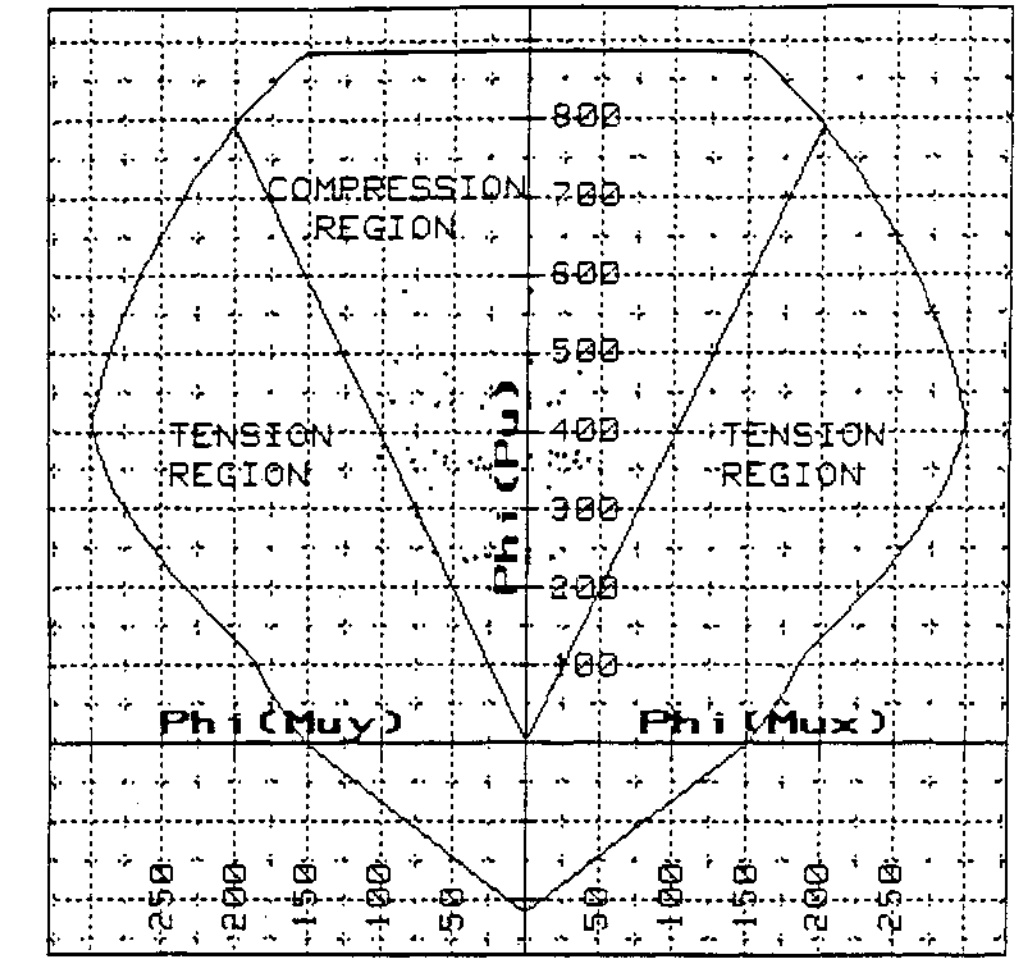
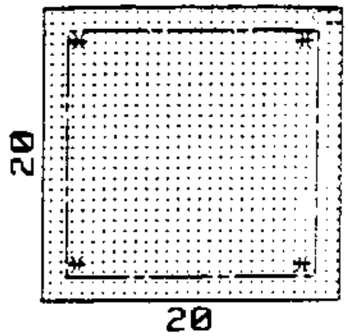
A6-4, D6-4
A4-4, D4-4





f´c= 4 KSI
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CORNER BARS
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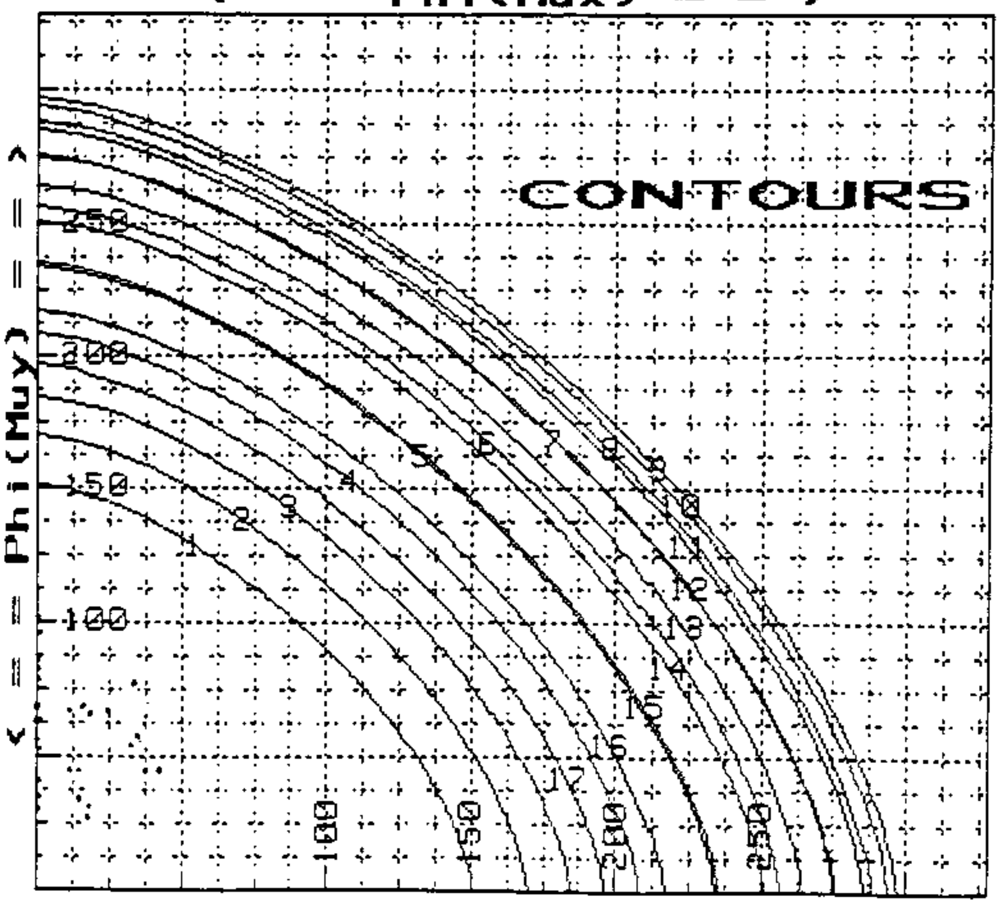
TOT. BARS= 4
Ast = 4
Rho = .01
KIPS & Ft. KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS

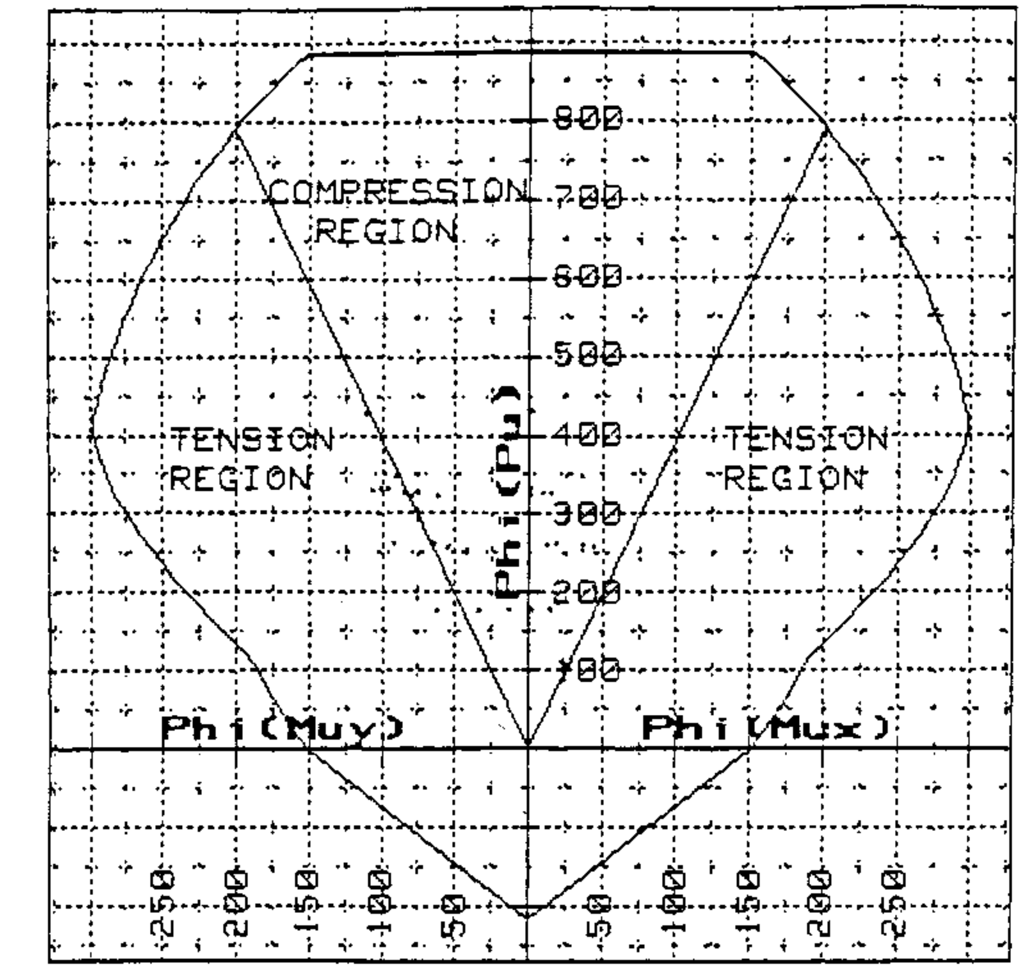
< = = Phi(Mux) = = >

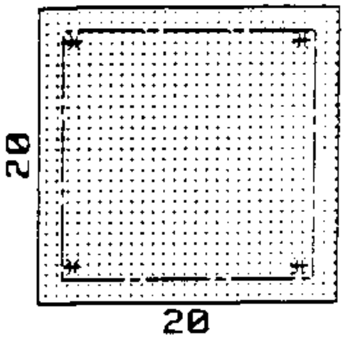


Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
3	100
4	150
5	200
6	250
7	300
8	350
9	400
10	450
11	500
12	550
13	600
14	650
15	700
16	750
17	800

A6-1, D5-1



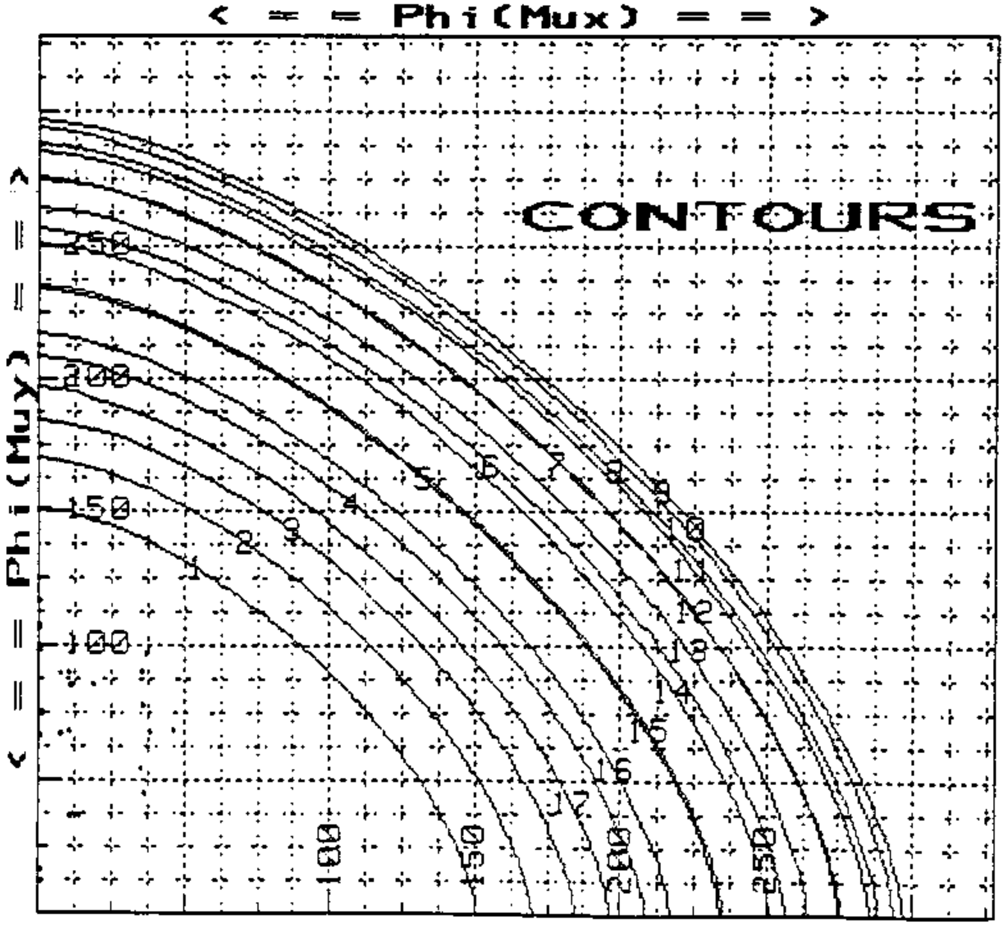


f'c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
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7	300
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12	550
13	600
14	650
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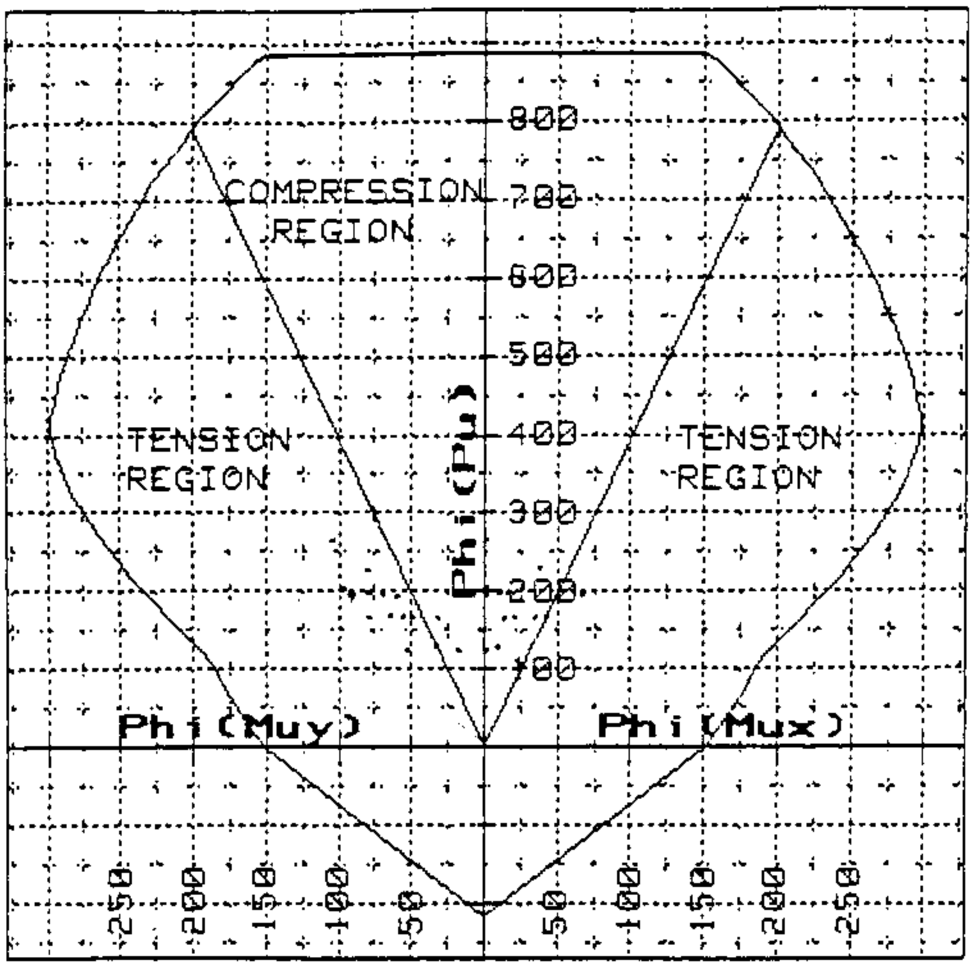
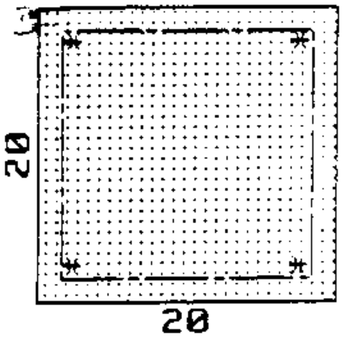
16	750
17	800



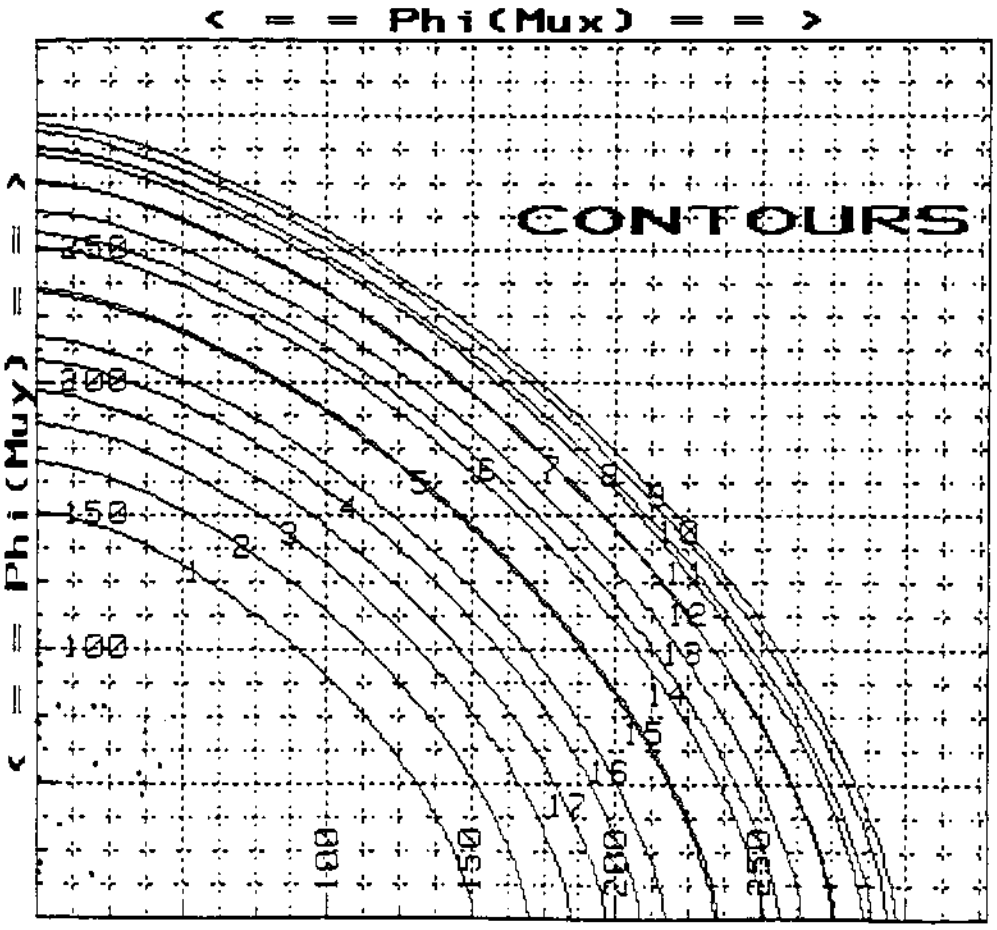


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
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Ix, Iy Steel
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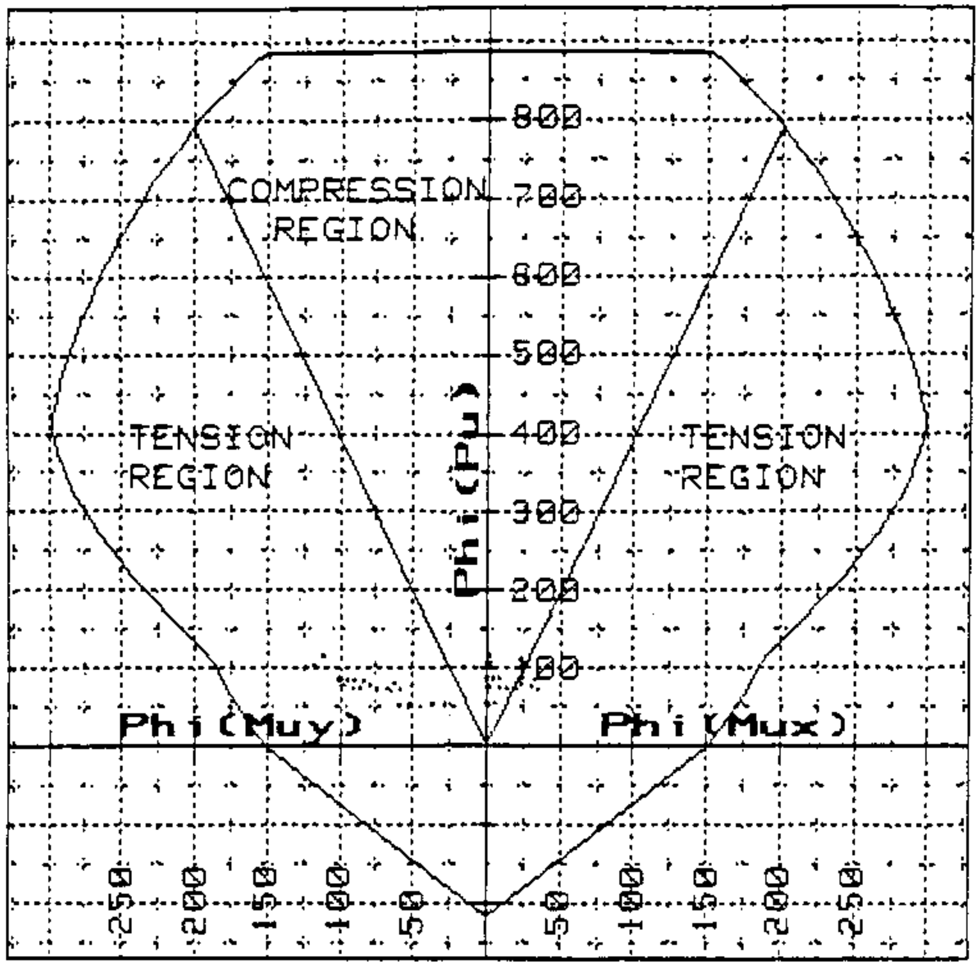

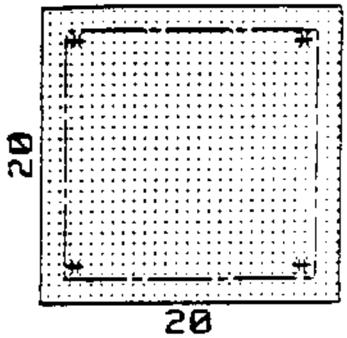

f´c= 4 KSI
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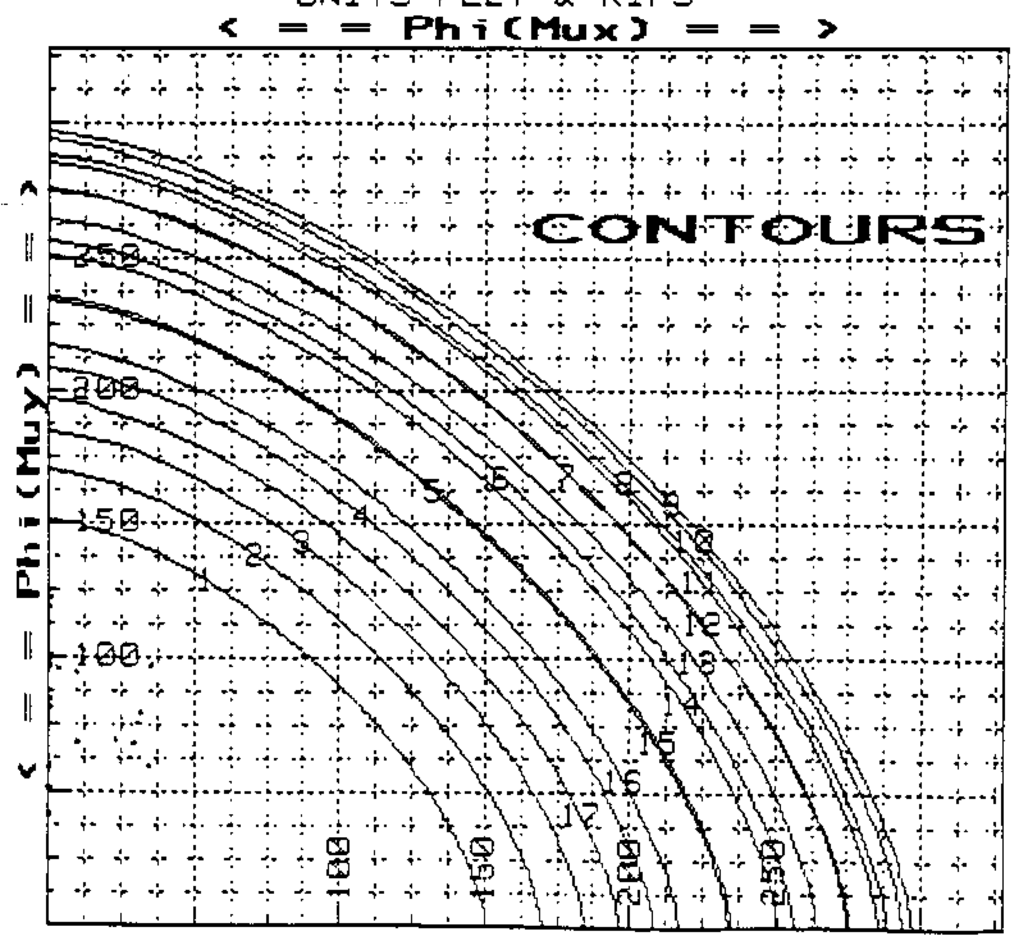
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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Ix, Iy Steel
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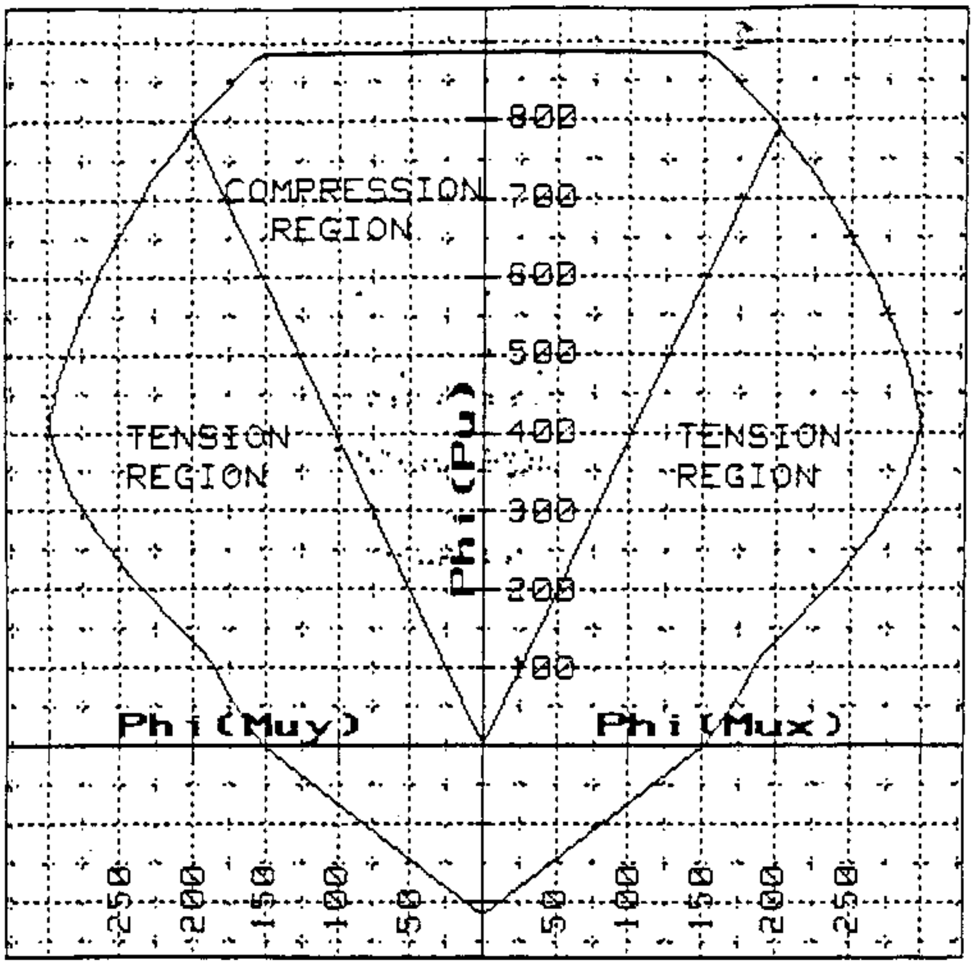

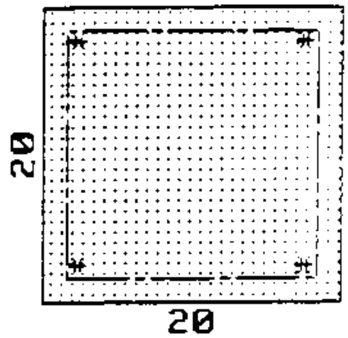

f'c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
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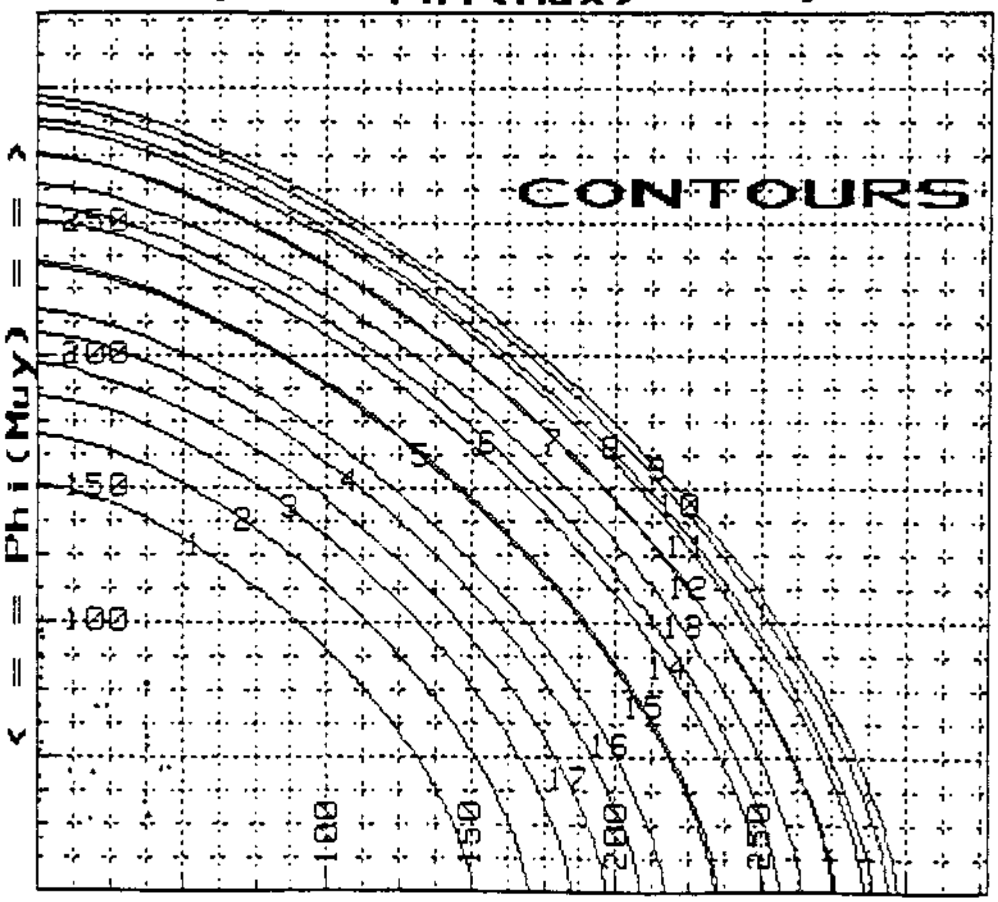
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS

< = = Phi(Mux) = = >



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5	200
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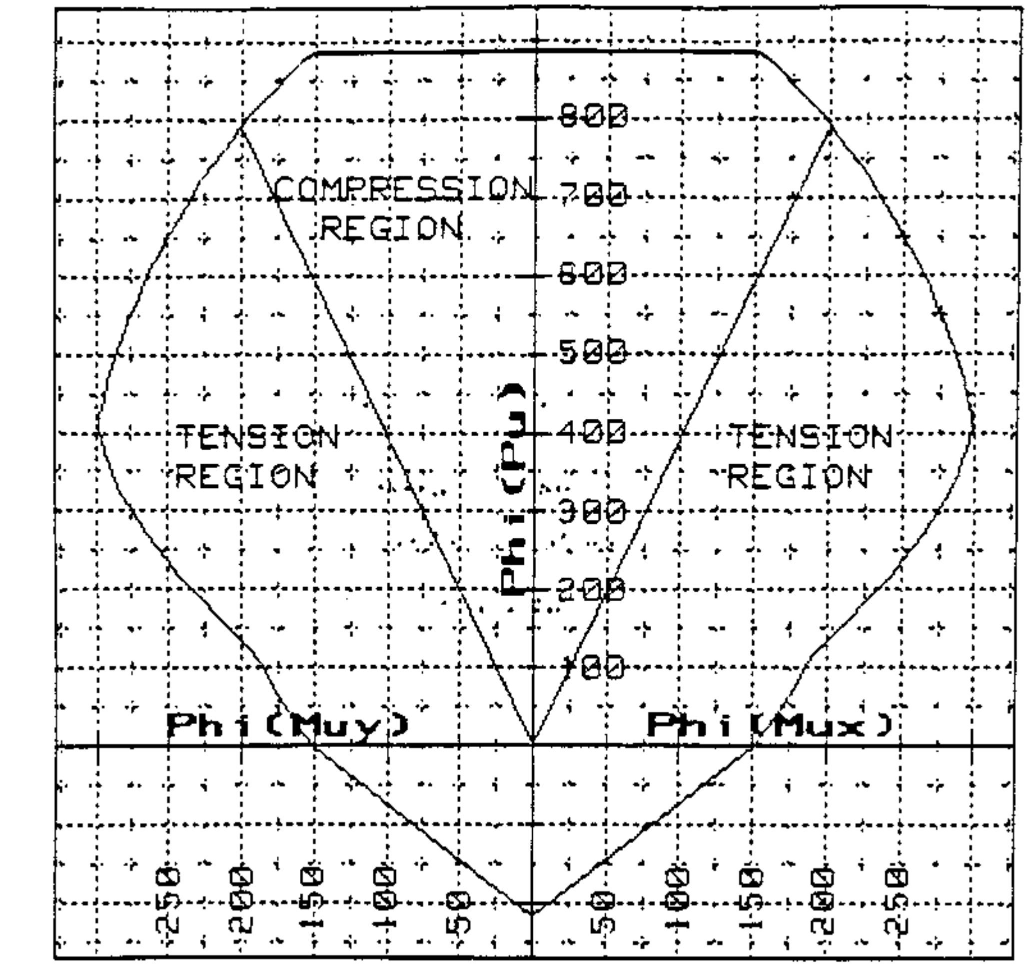
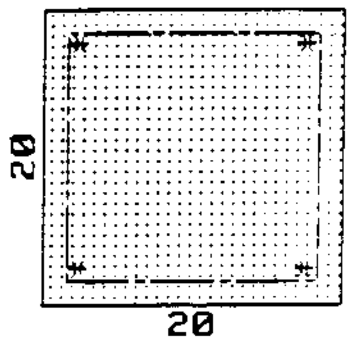
11	500
12	550
13	600
14	650
15	700
16	750
17	800

A8-L, 754



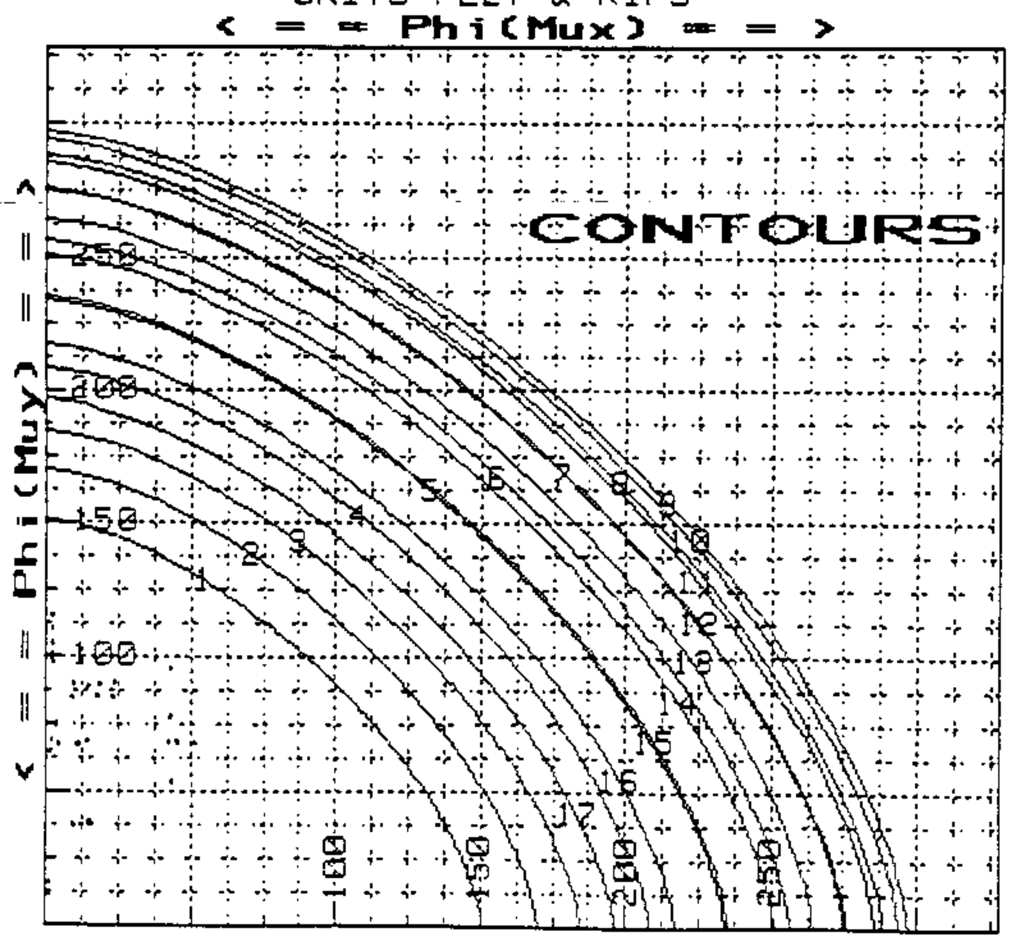


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
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4	150
5	200
6	250
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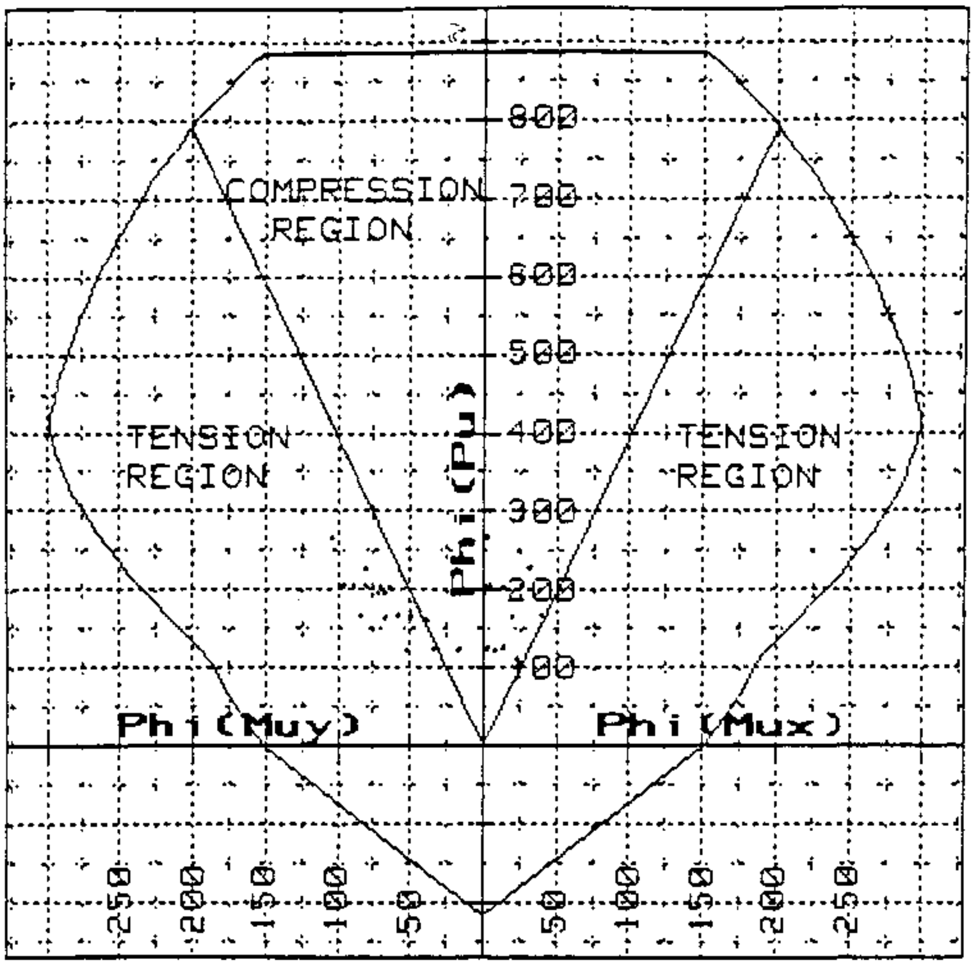
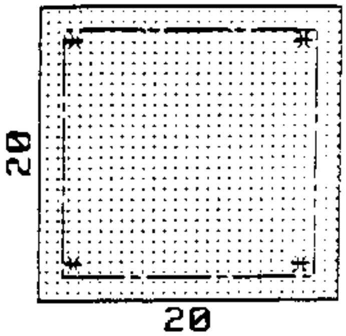
10	450
11	500
12	550
13	600
14	650
15	700
16	750
17	800

AF-2, DB-2



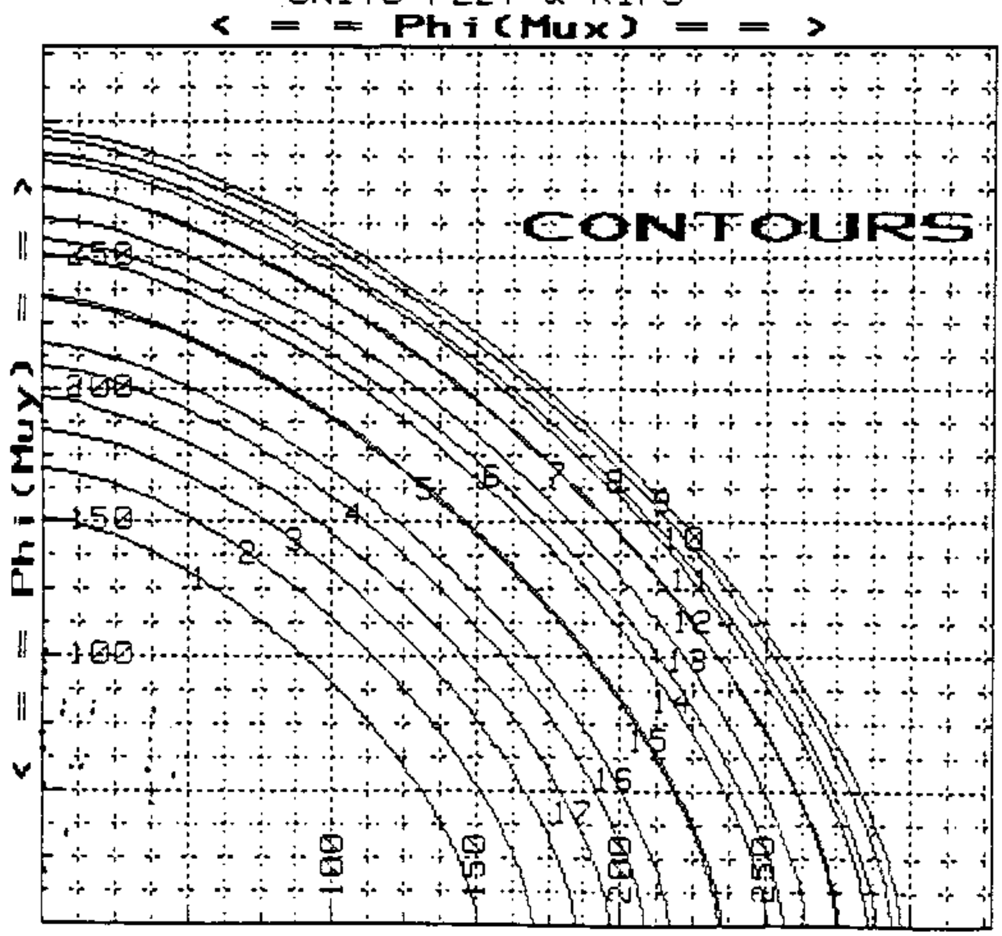


f'c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
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5	200
6	250
7	300
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13	600
14	650
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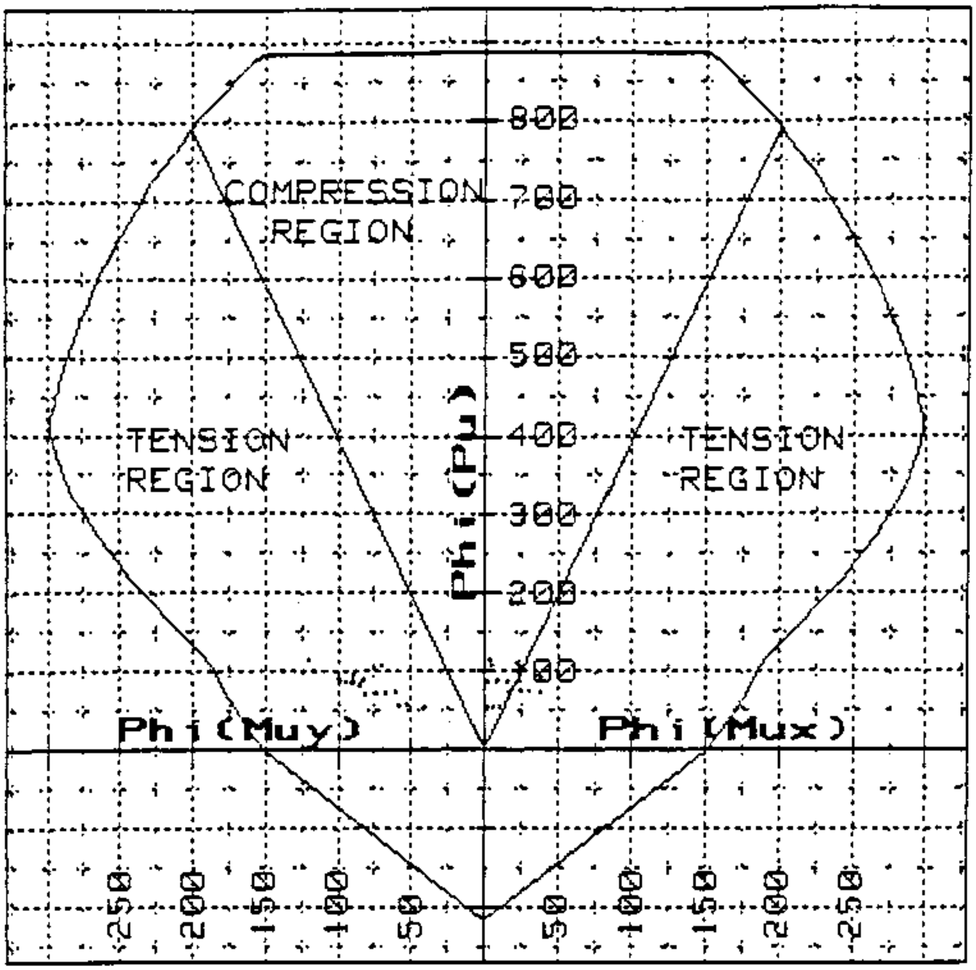
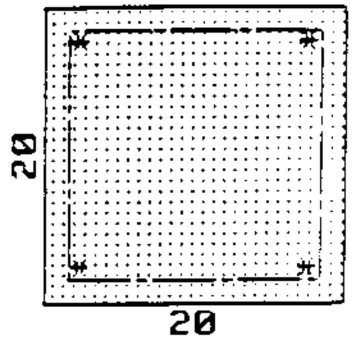


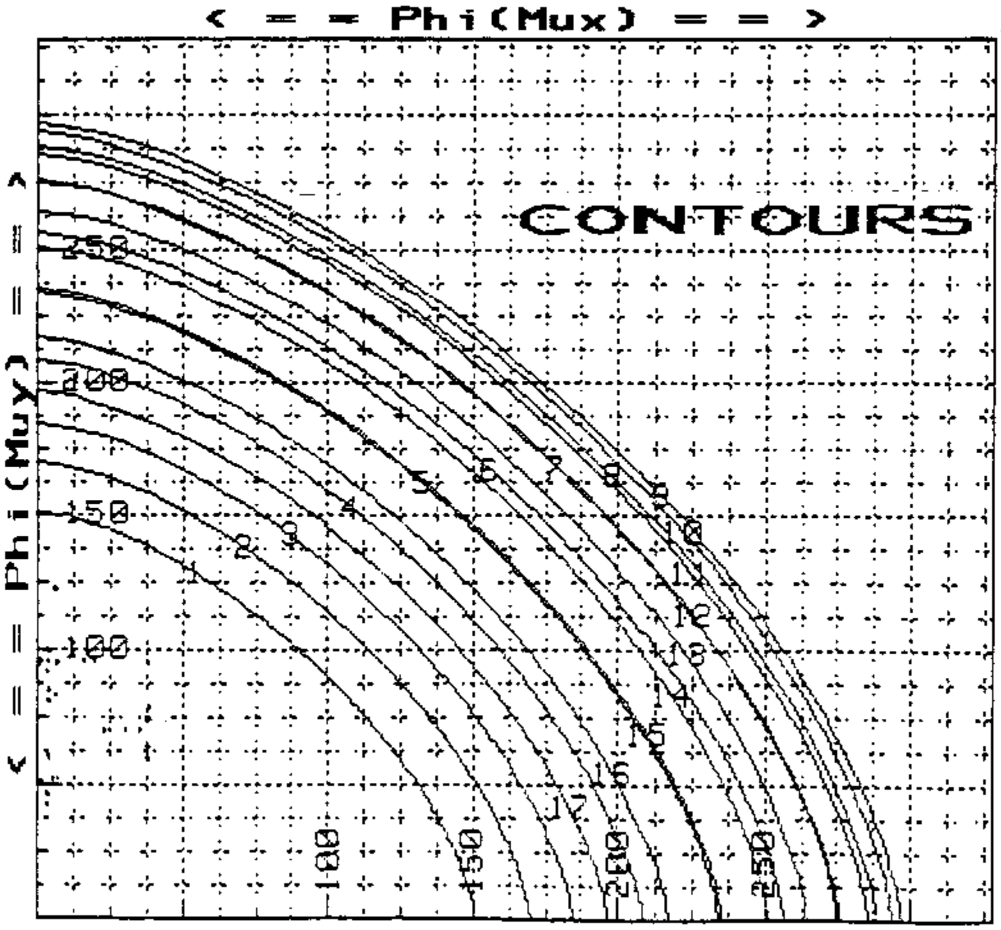


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
3	100
4	150
5	200
6	250
7	300
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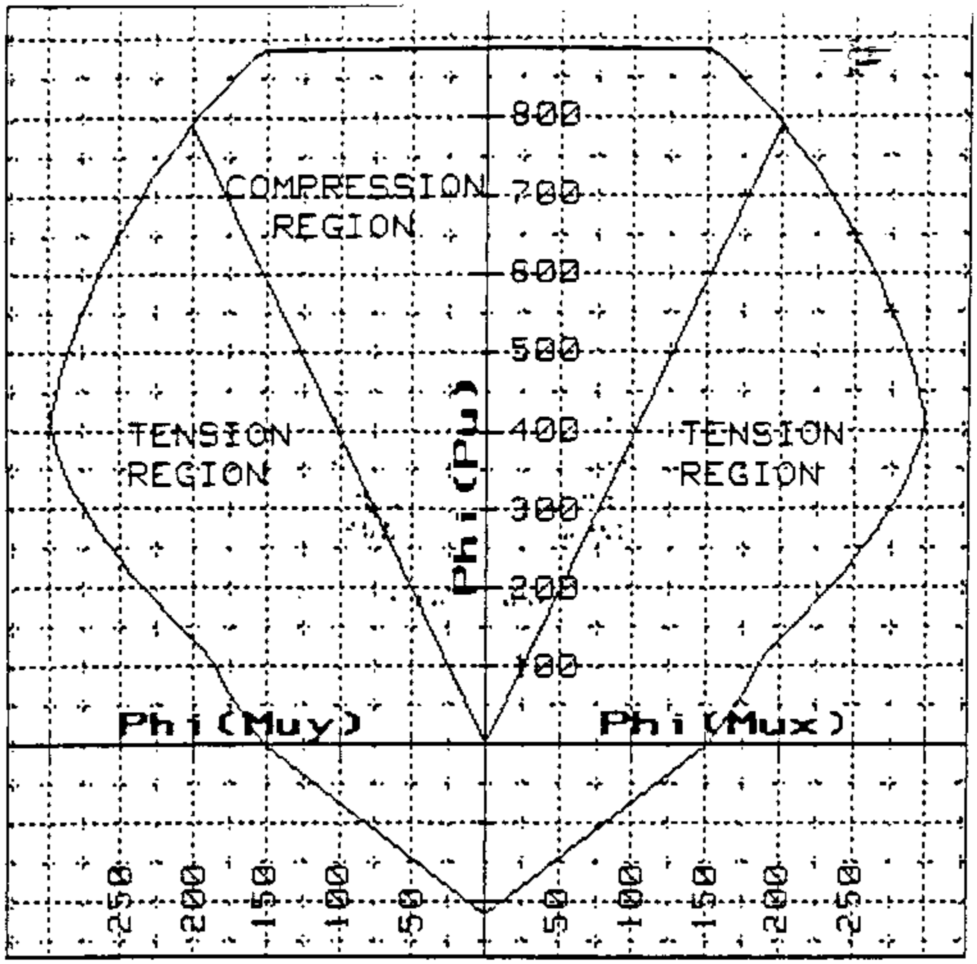
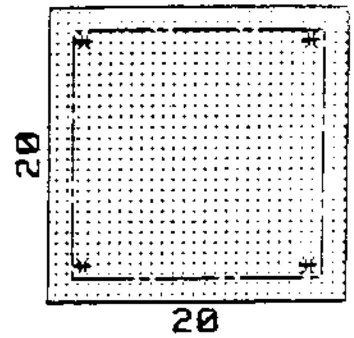
11	500
12	550
13	600
14	650
15	700
16	750
17	800

AB-4, 12B-4



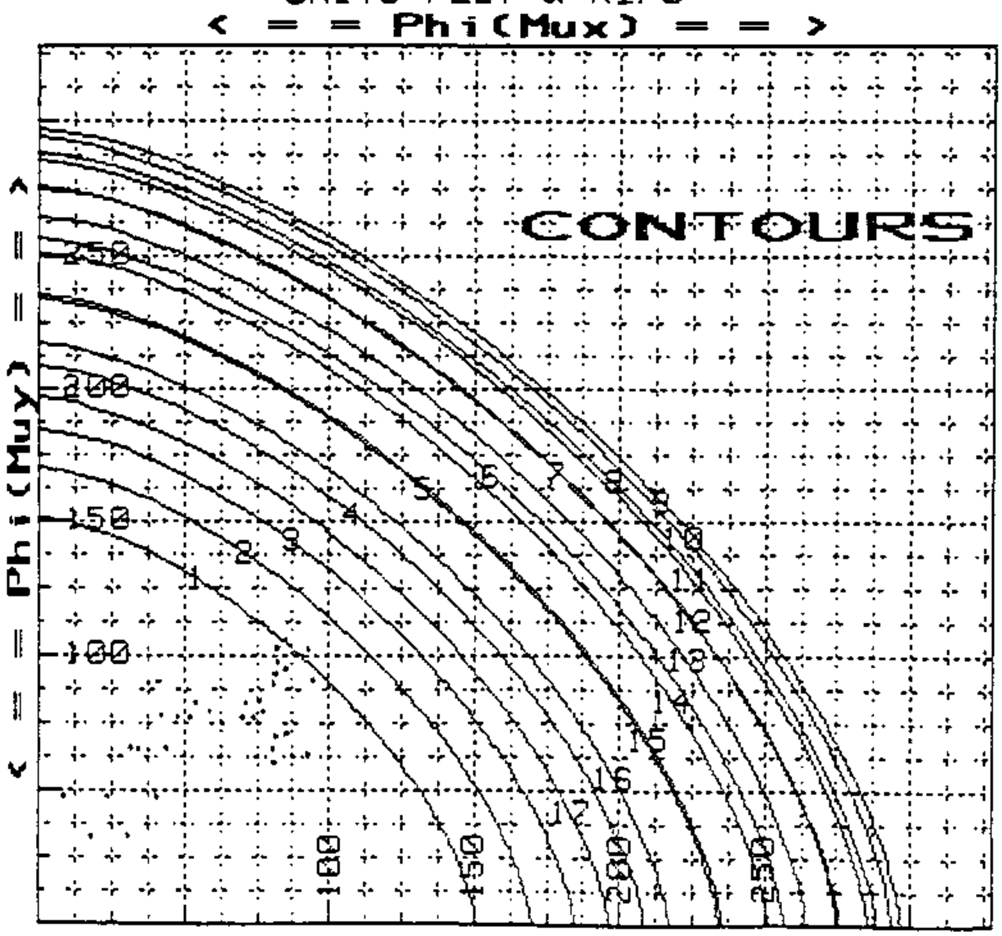


f'c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
3	100
4	150
5	200
6	250
7	300
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11	500
12	550
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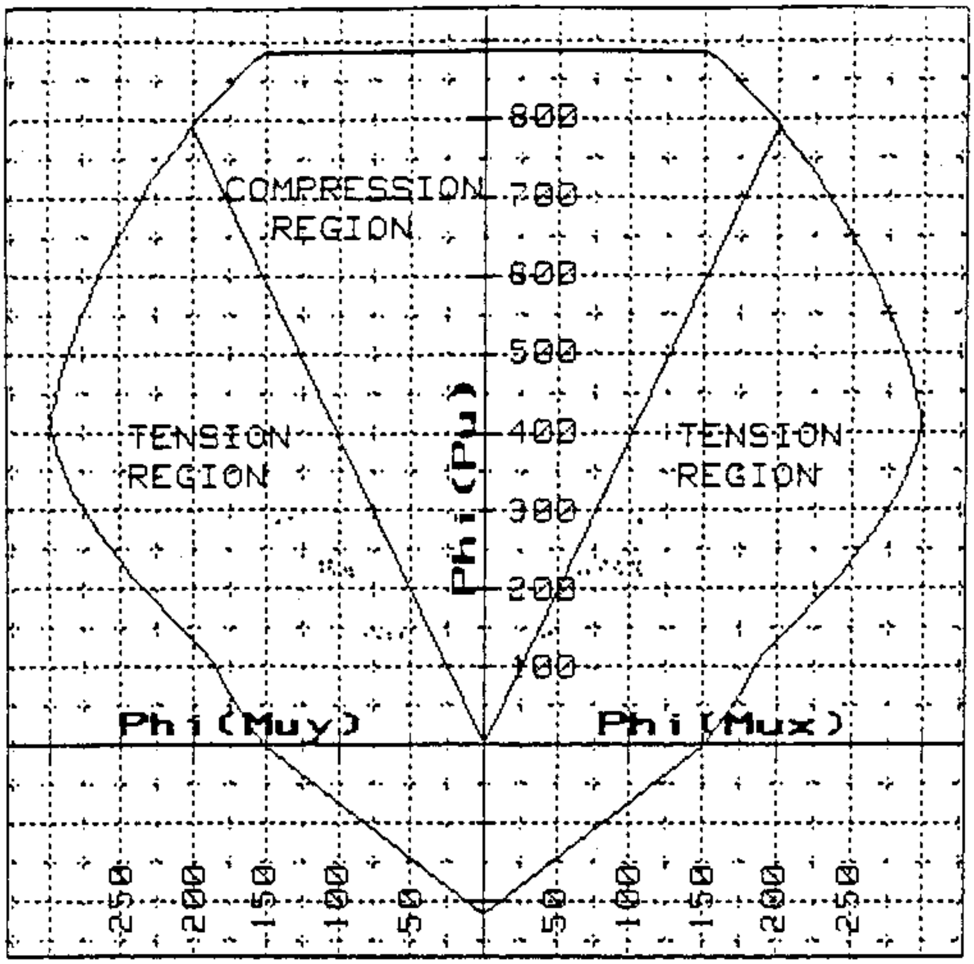
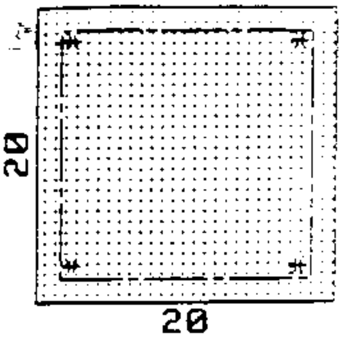
14	650
15	700
16	750
17	800

A1-1, D9-1



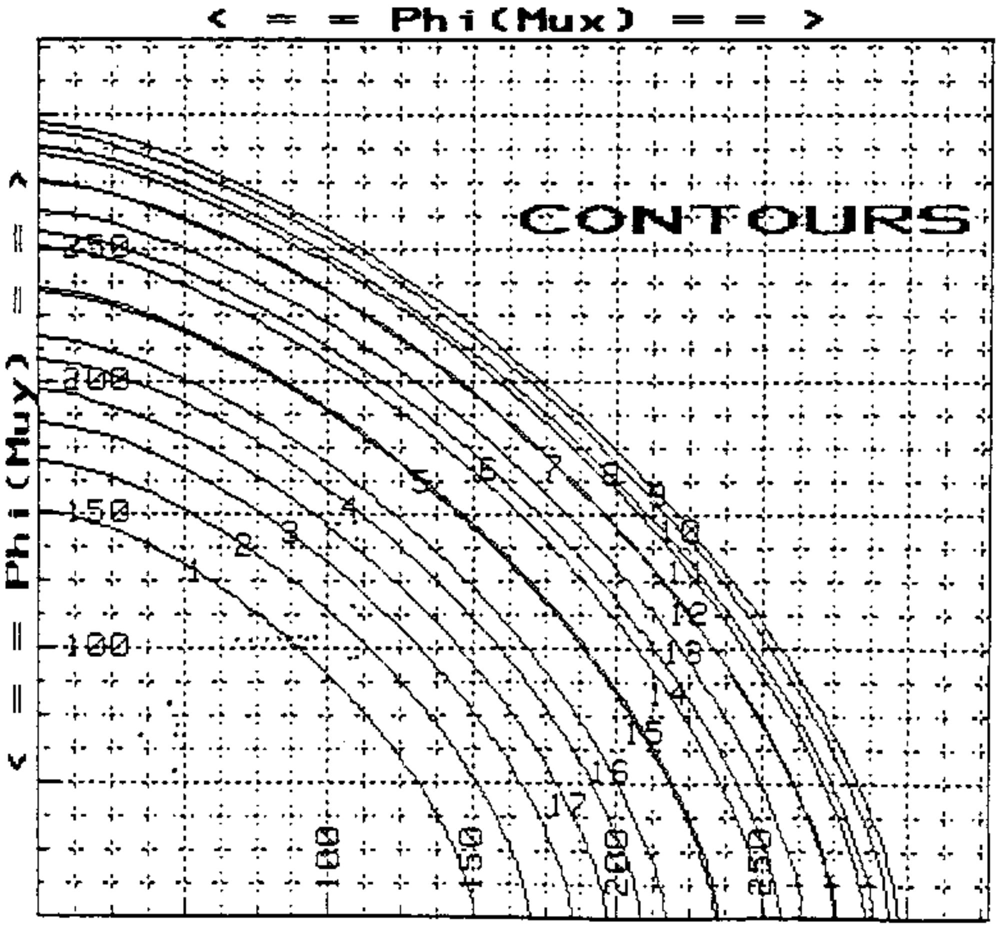


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
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3	100
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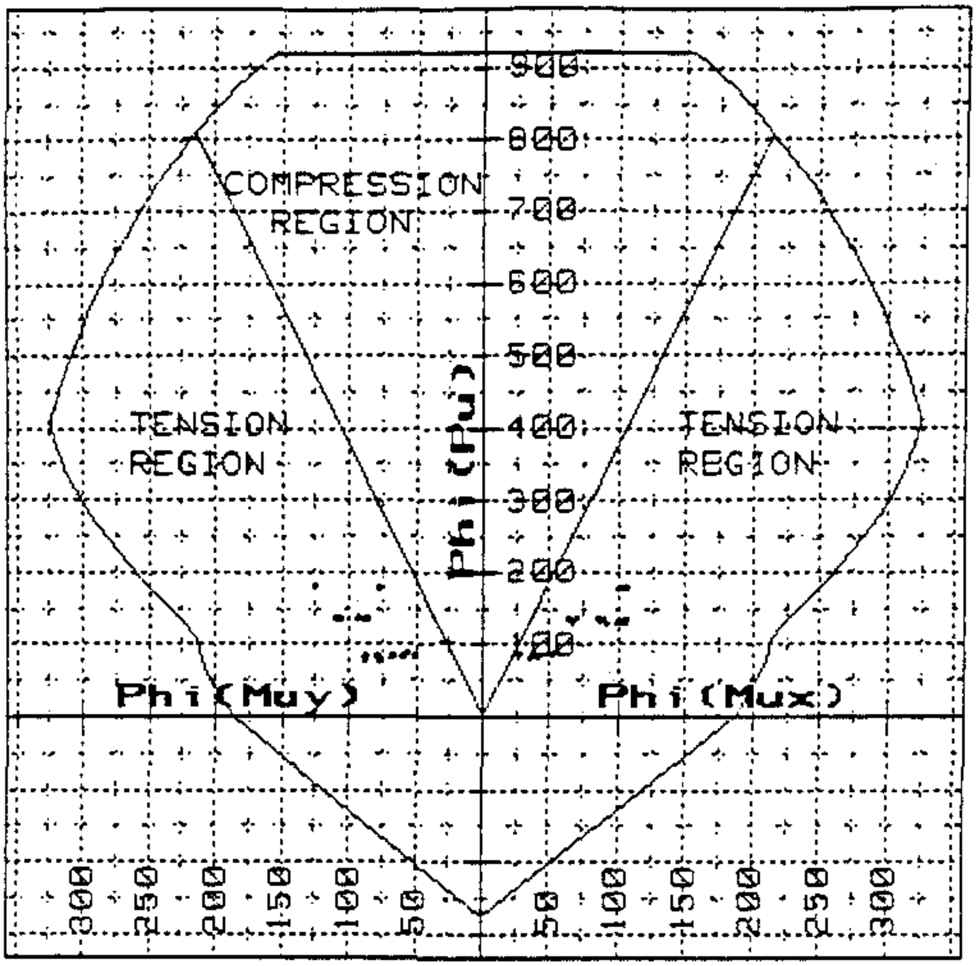
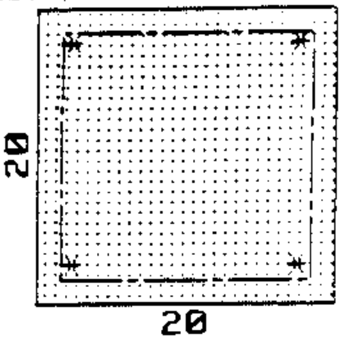
16	750
17	800

A4-2, b2-c





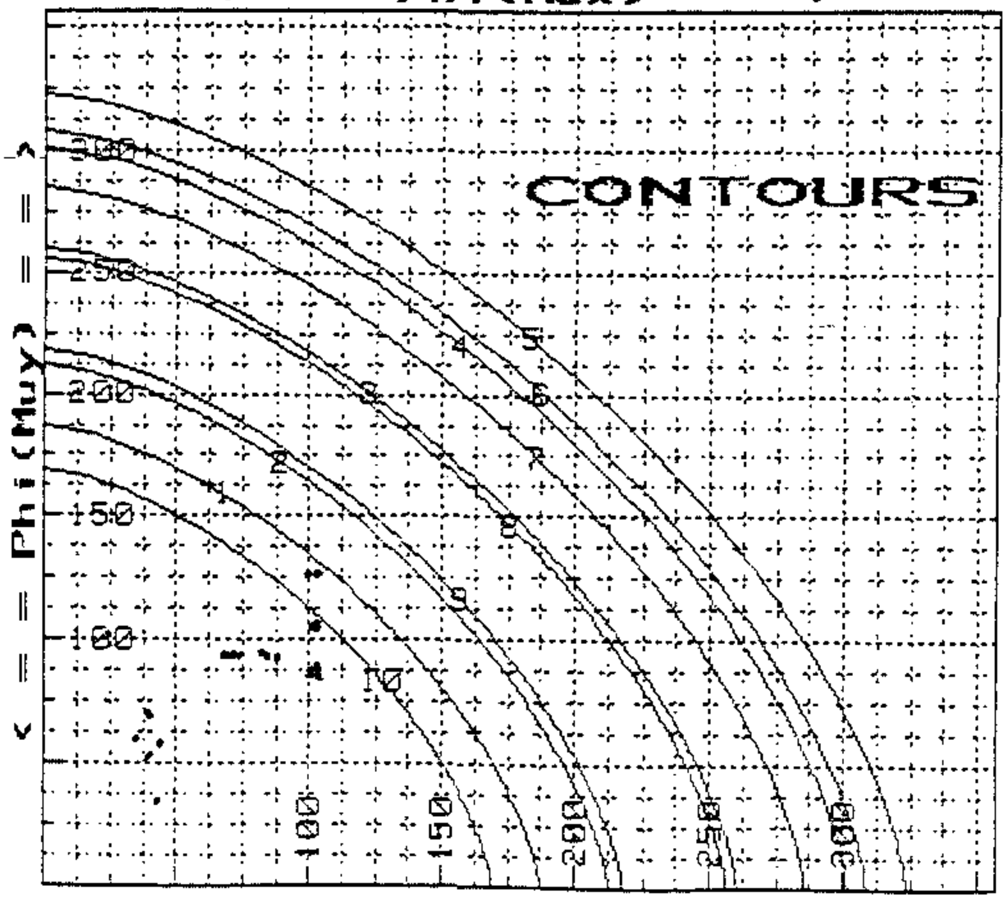
f´c= 4 KSI
~~Fy = 60 KSI~~
3 TIES
@ 18 O.C.
CORNER BARS
4 # 10
TOT.BARS= 4
Ast = 5.08
Rho = .0127
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS

< = = Phi(Mux) = = >



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Iy= 13333.33
Ix, Iy Steel
Isex= 284.98
Isey= 284.98

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3	200
4	300
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6	500
7	600
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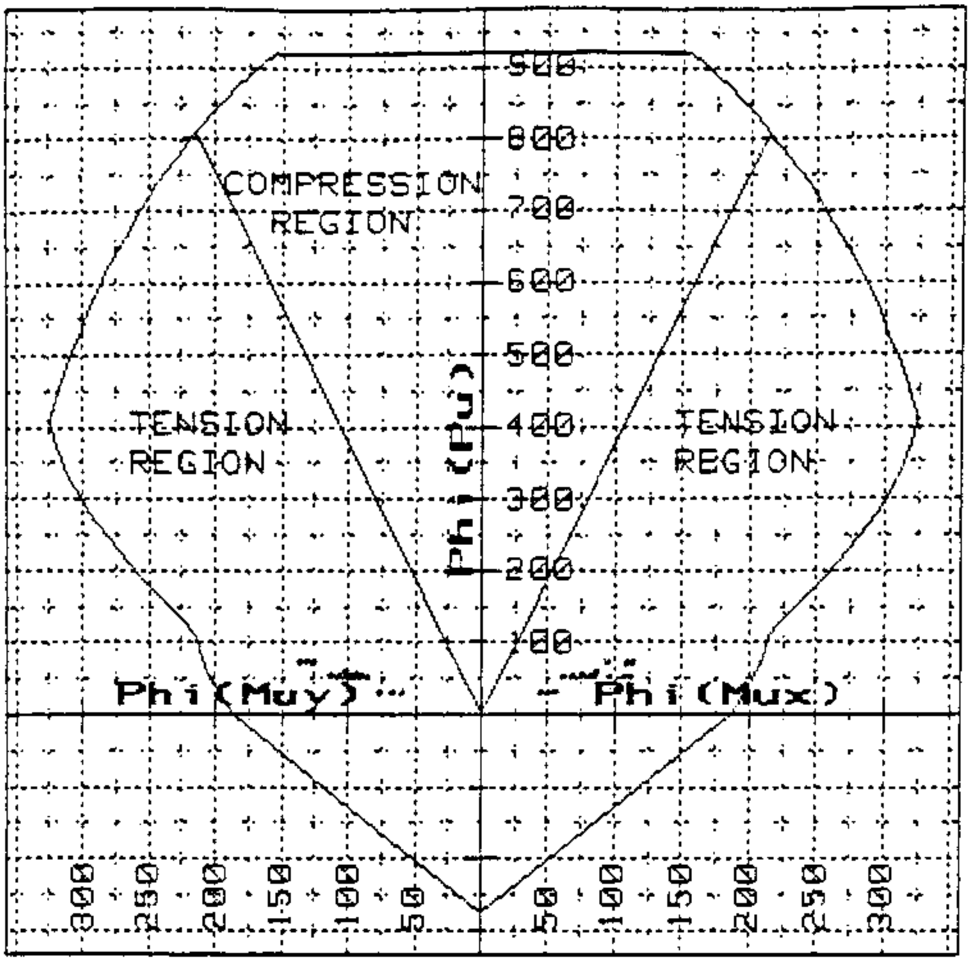
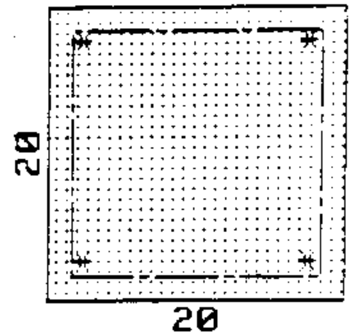
10	900



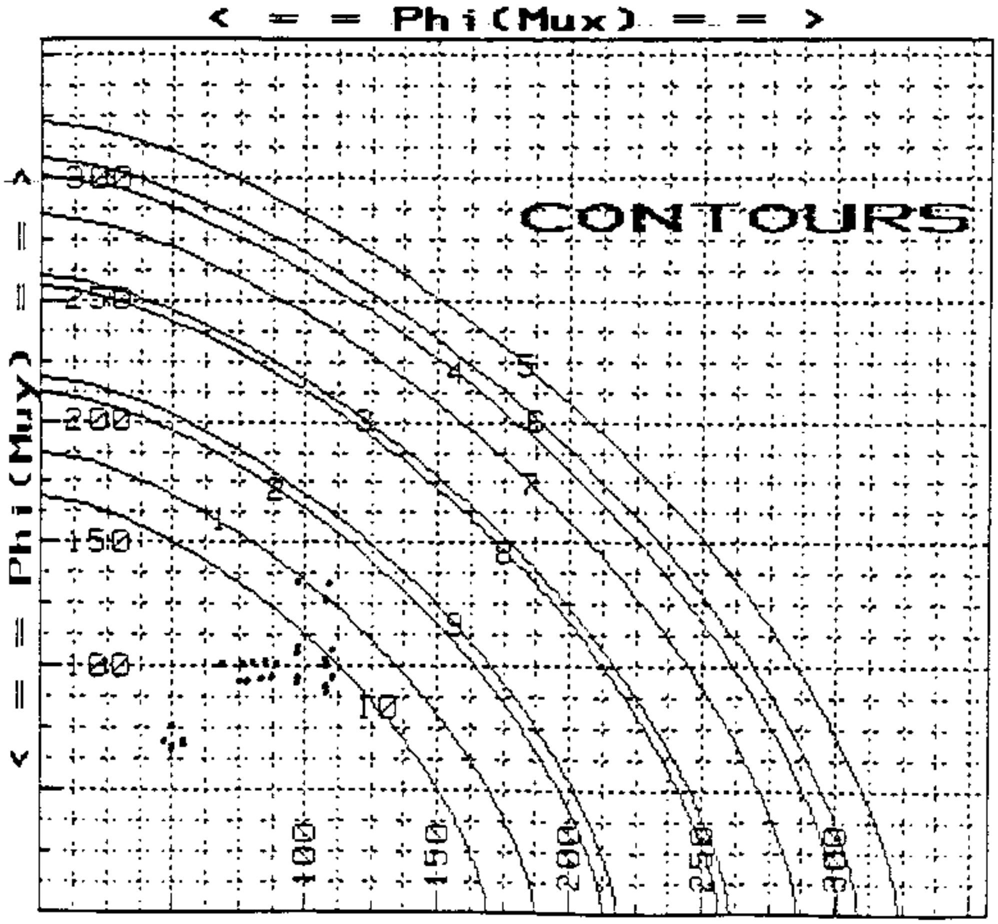


f'c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 10
TOT. BARS= 4
Ast = 5.08
Rho = .0127
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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Ix, Iy Steel
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LINE	Pu
1	0
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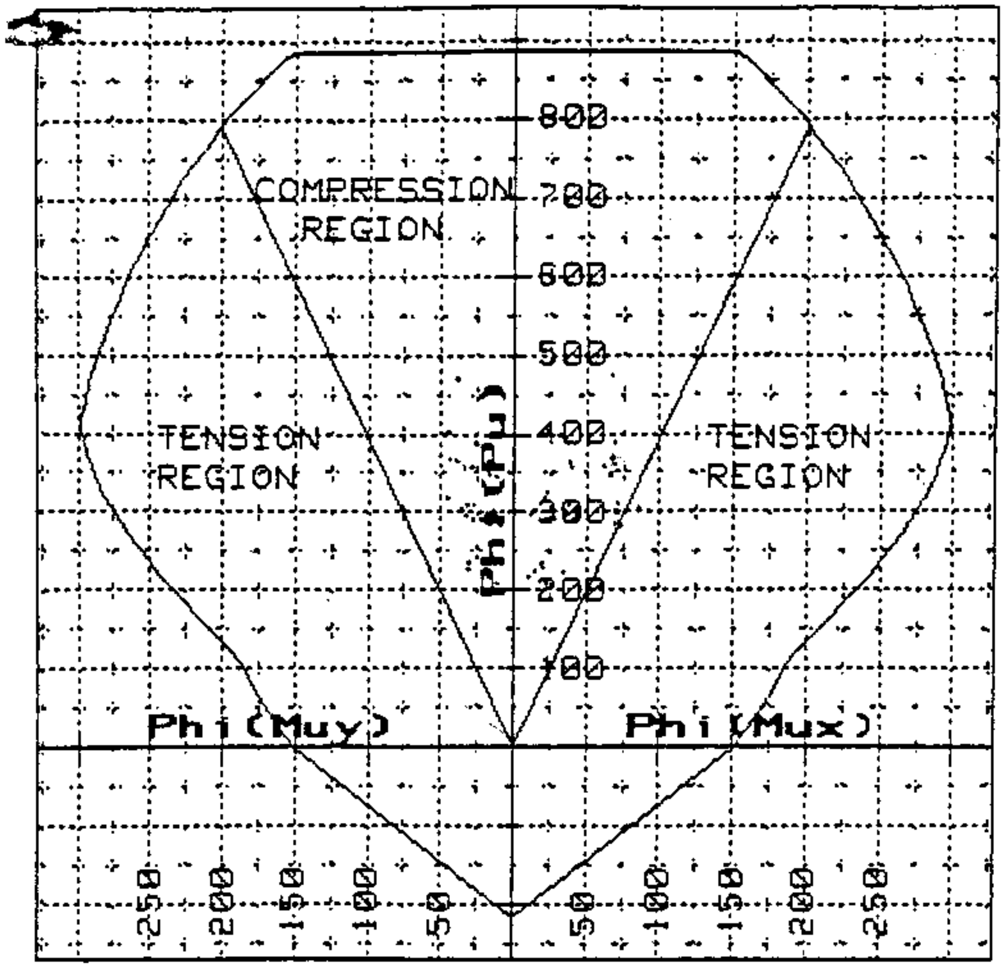
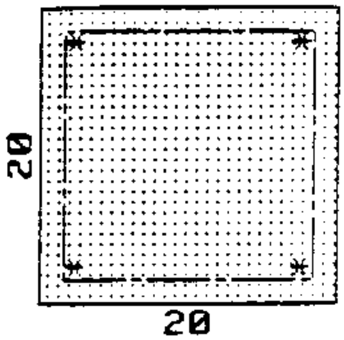
A9-4, B9-4



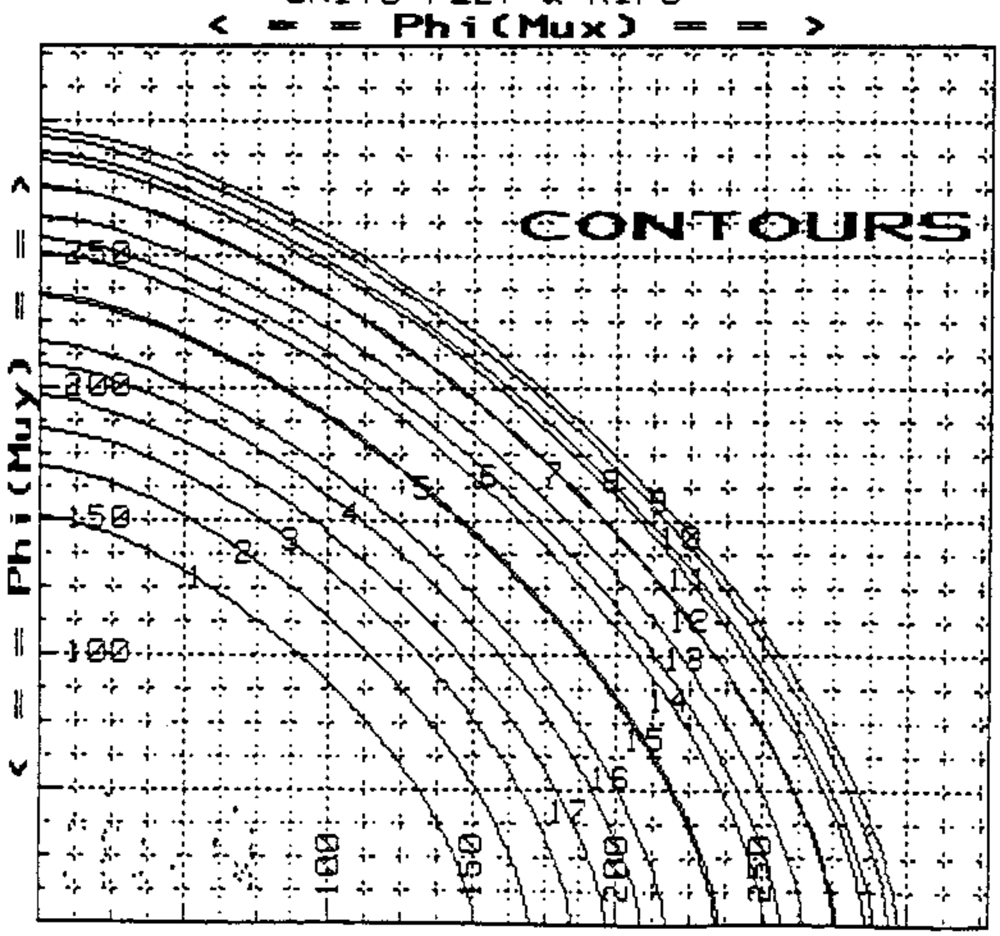


f'c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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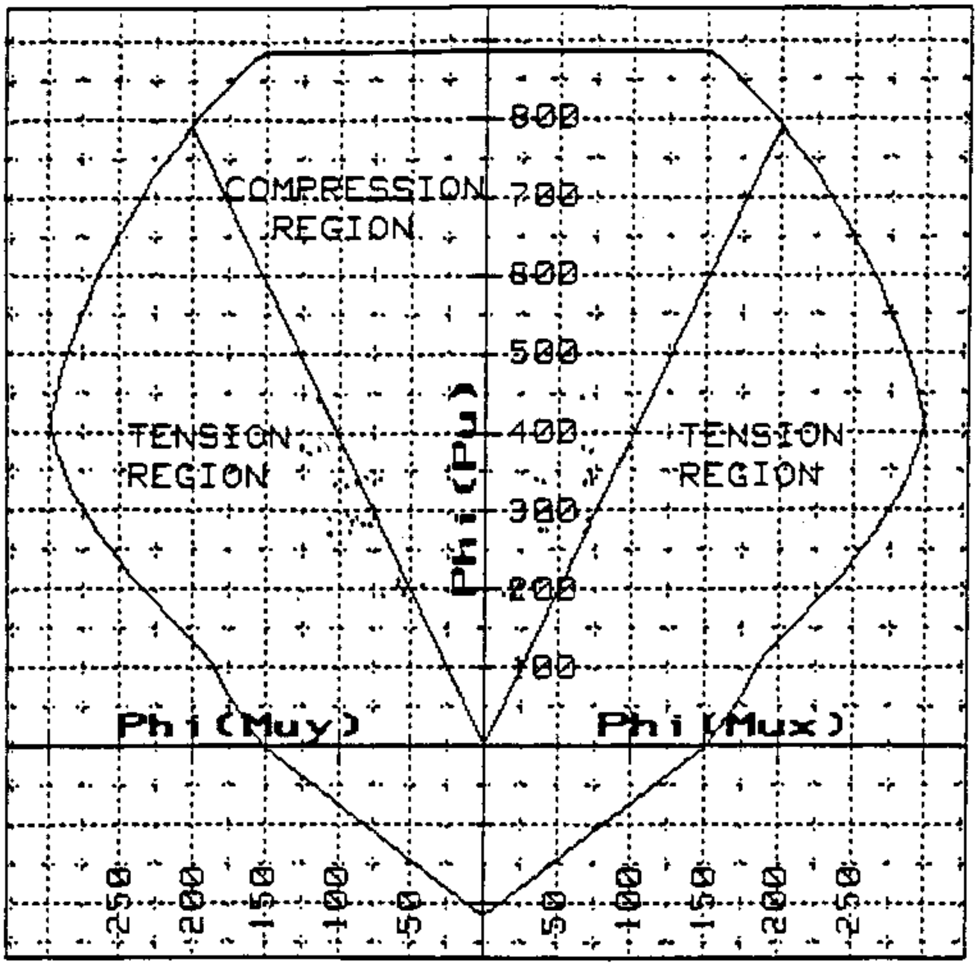

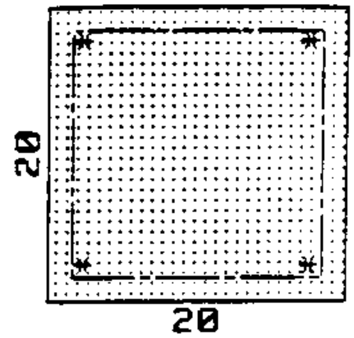

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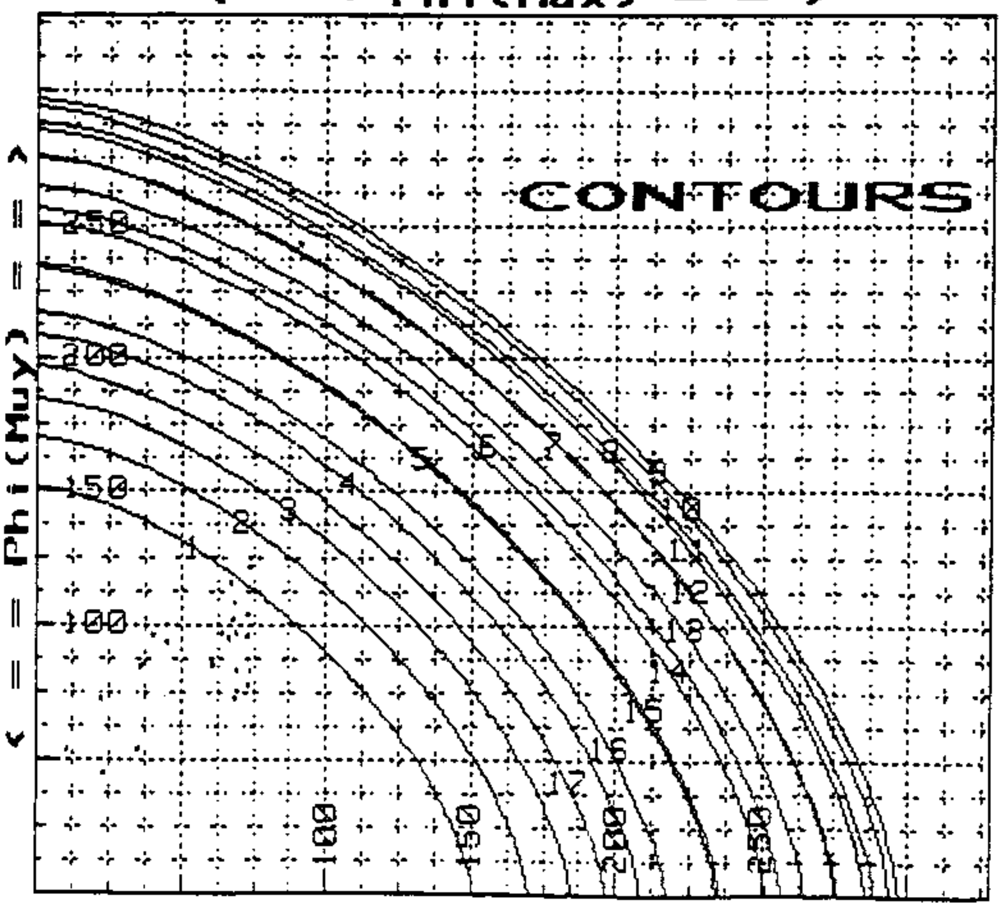
PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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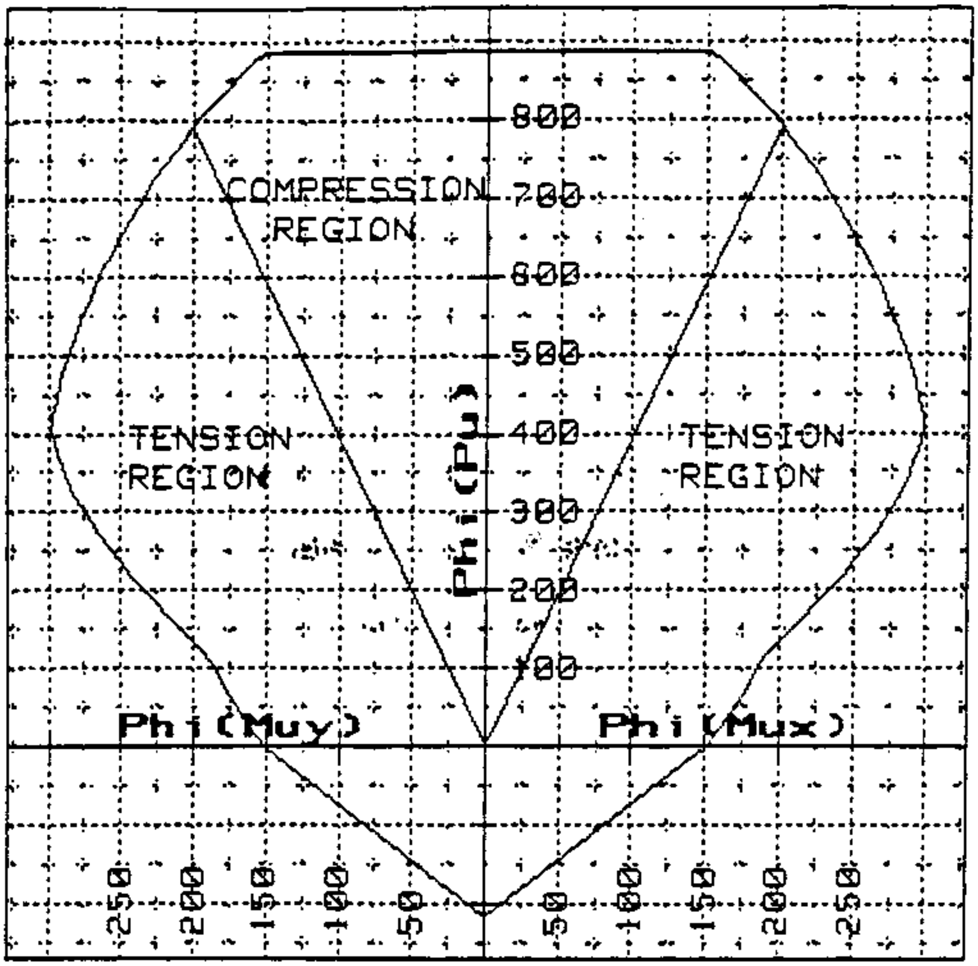

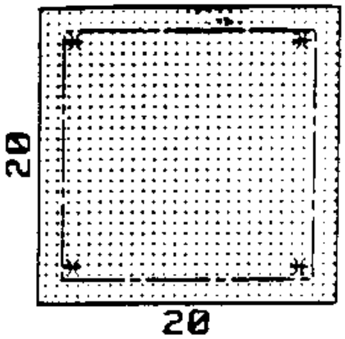

f´c= 4 KSI
Fy = 60 KSI
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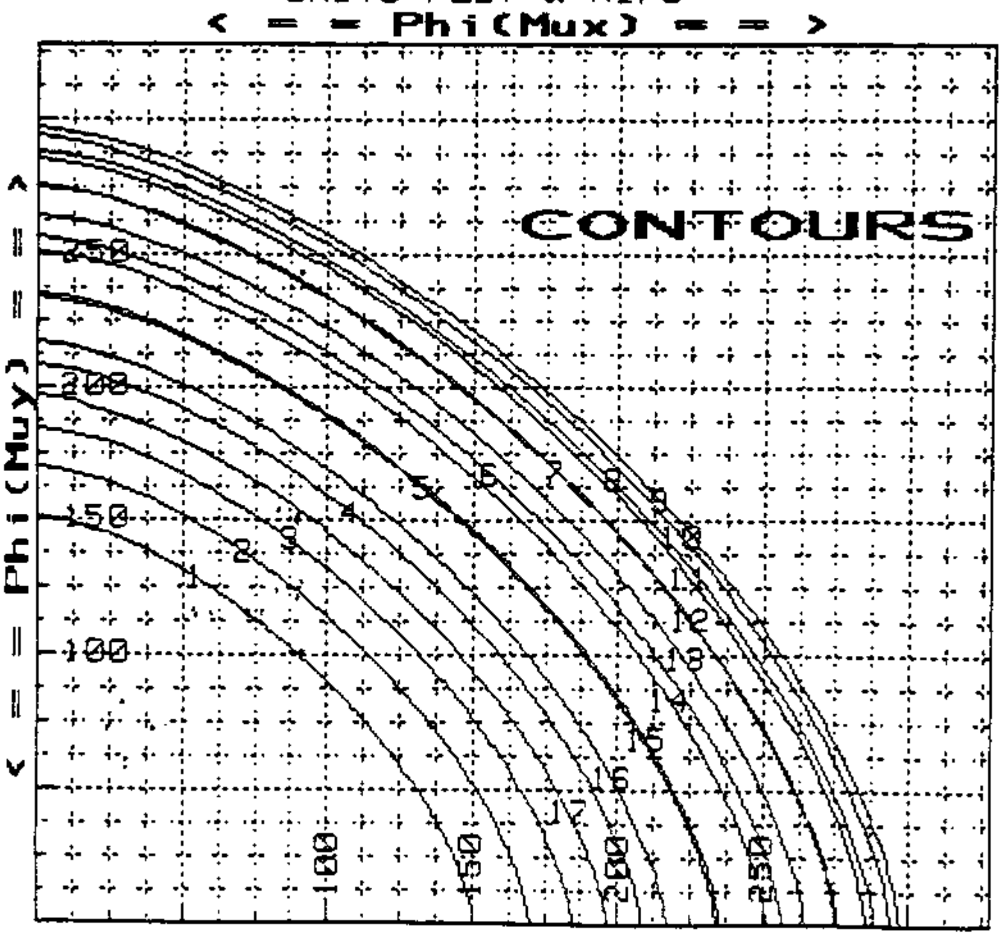
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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Ix, Iy Steel
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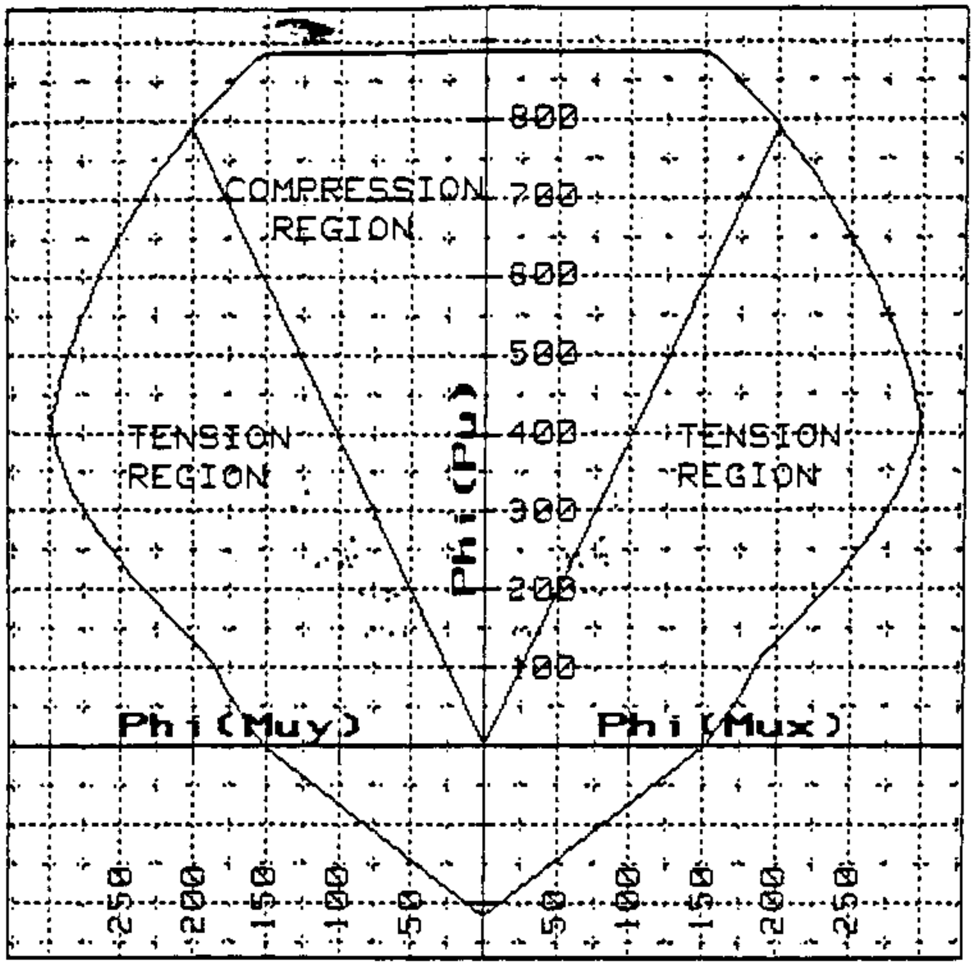
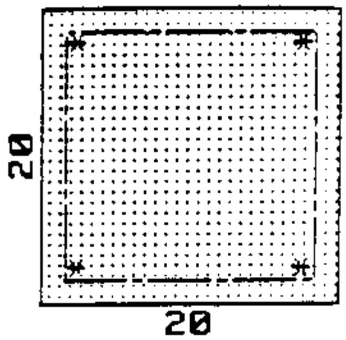
16	750
17	800



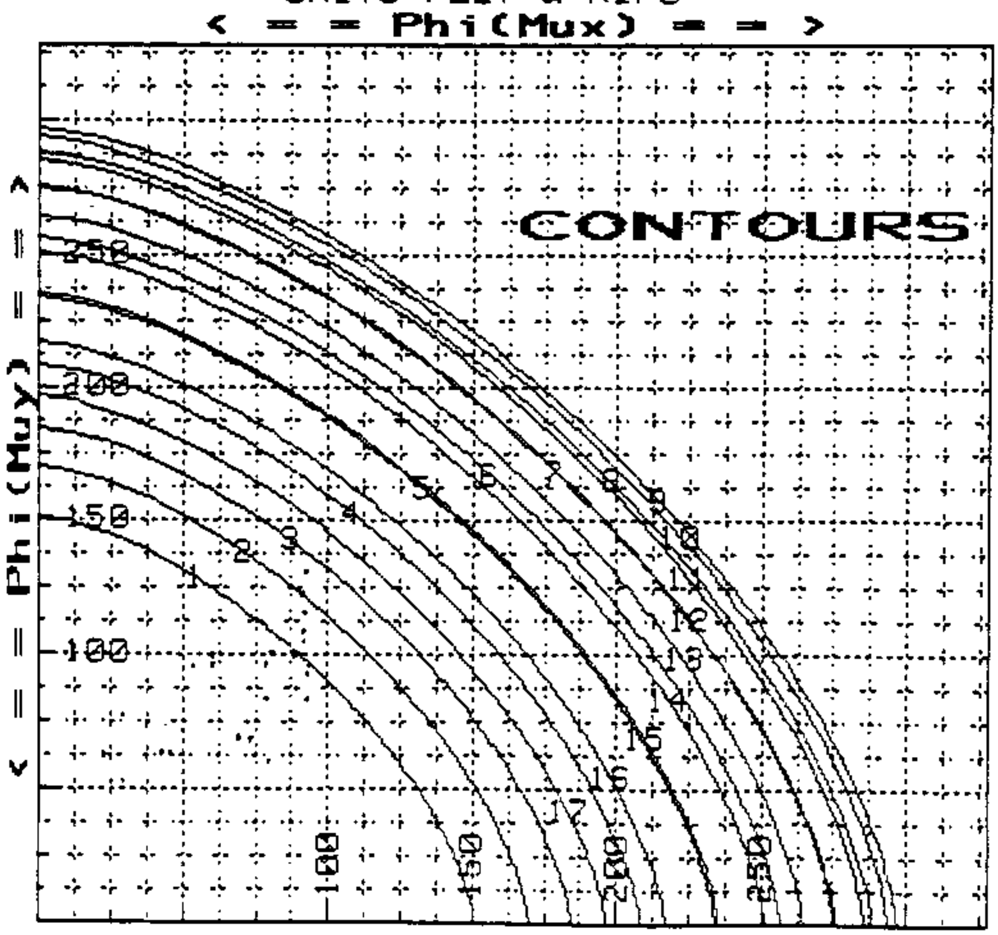


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
Iy= 13333.33
Ix, Iy Steel
Isex= 228.67
Isey= 228.67

LINE	Pu
1	0
2	50
3	100
4	150
5	200
6	250
7	300
8	350
9	400
10	450
11	500
12	550
13	600
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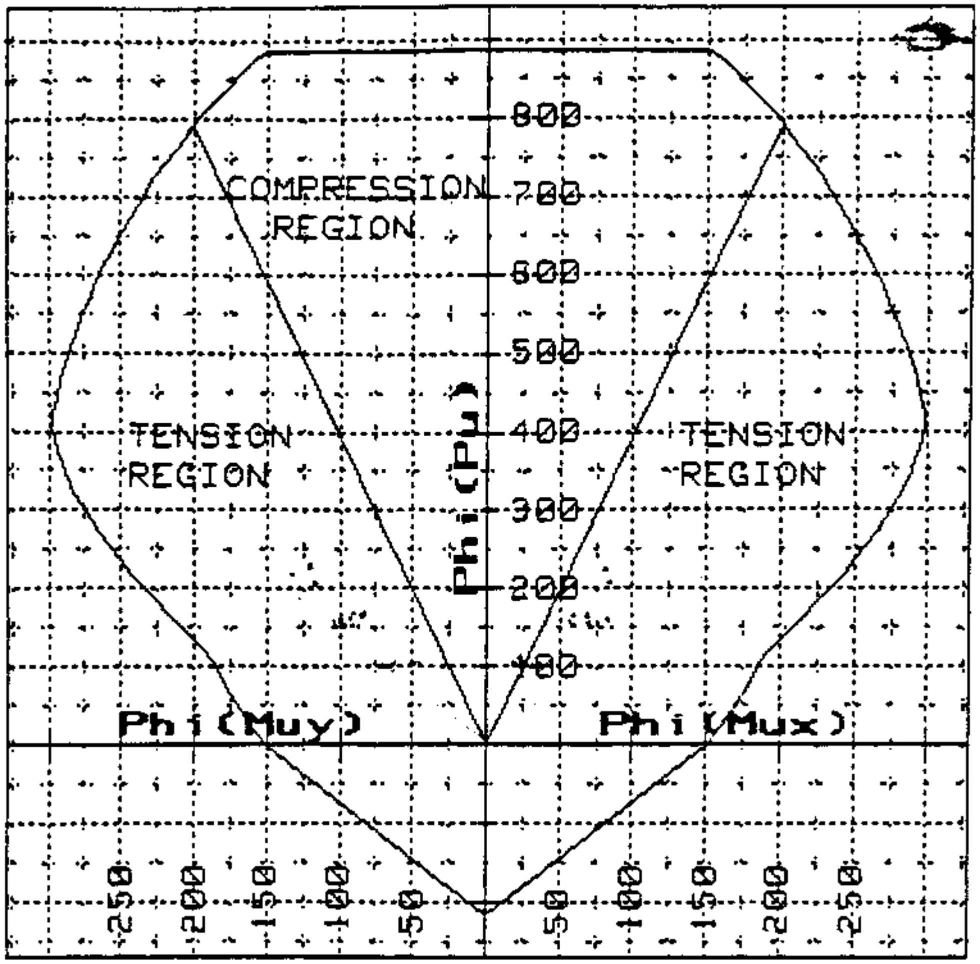

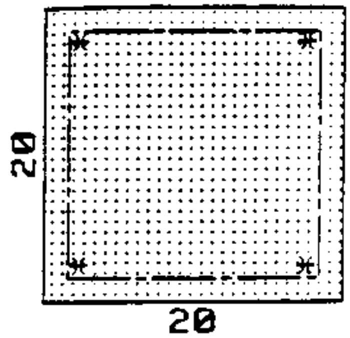

f´c= 4 KSI
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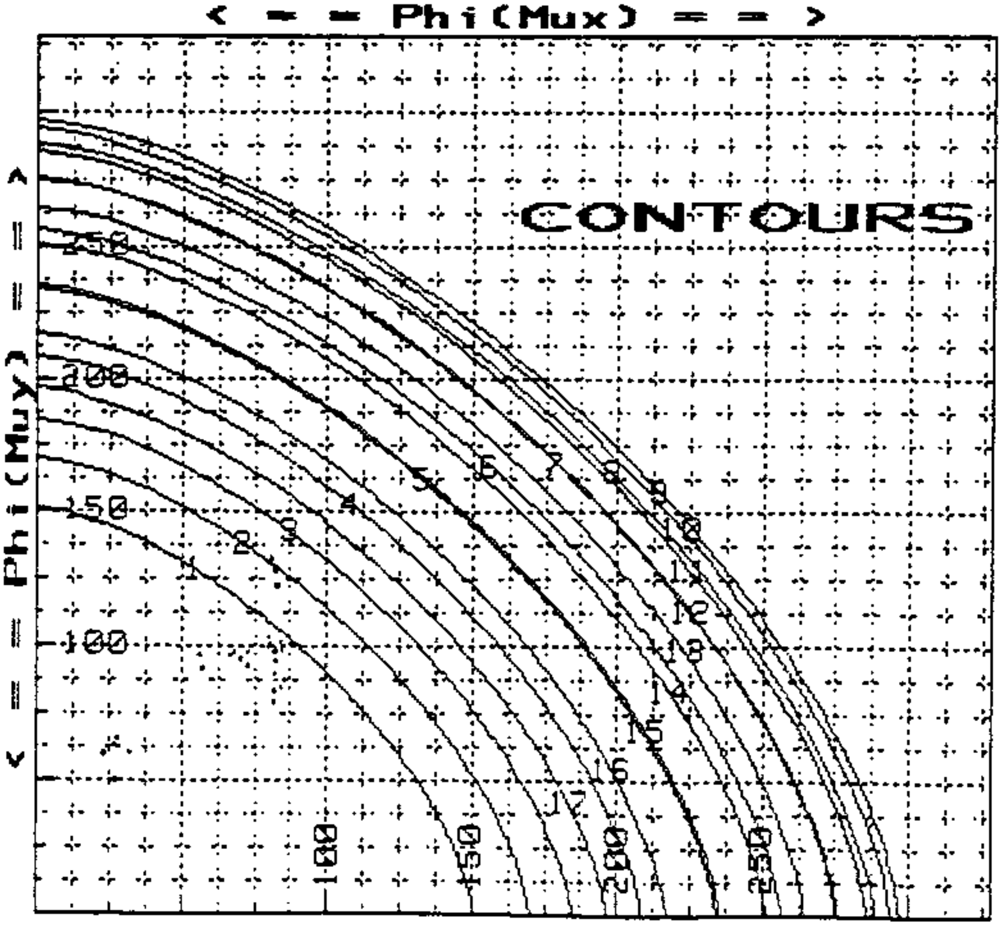
PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS

< = = Phi(Mux) = = >



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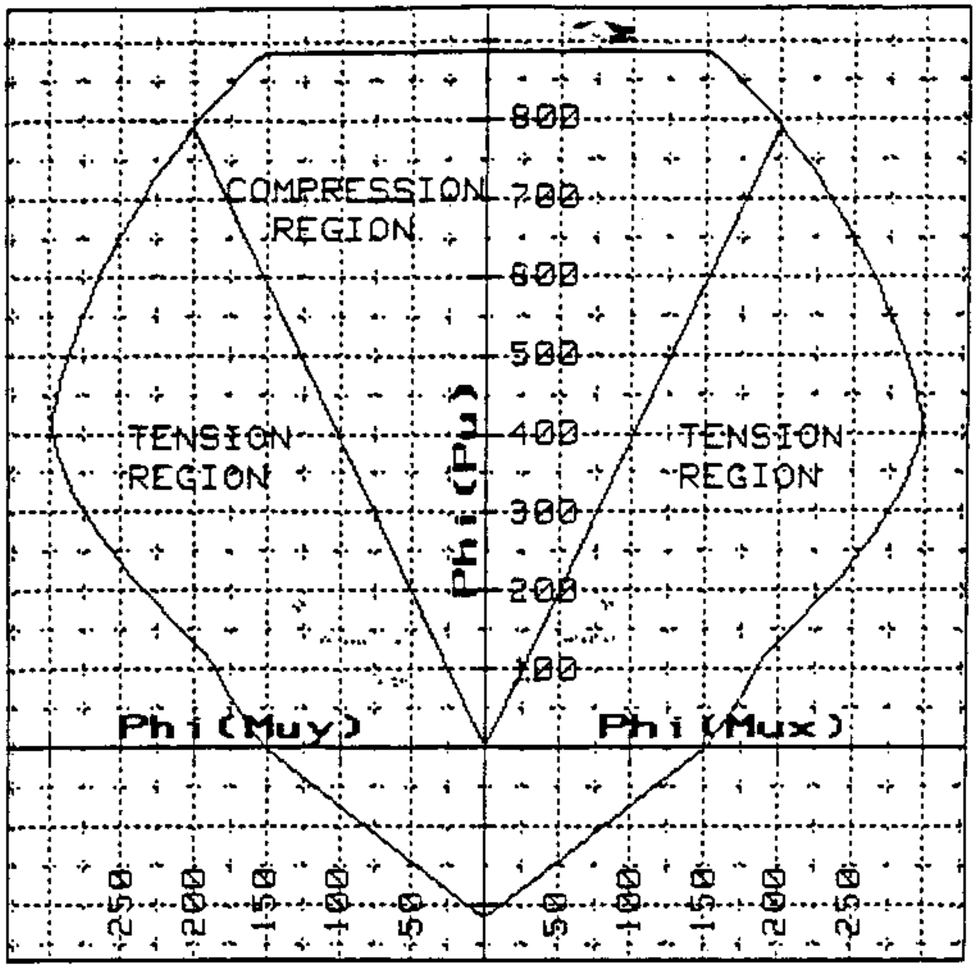
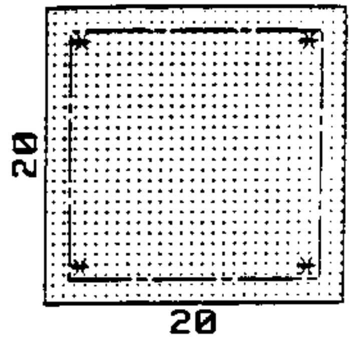
12	550
13	600
14	650
15	700
16	750
17	800

B1-32 C1-32





f'c= 4 KSI
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CORNER BARS
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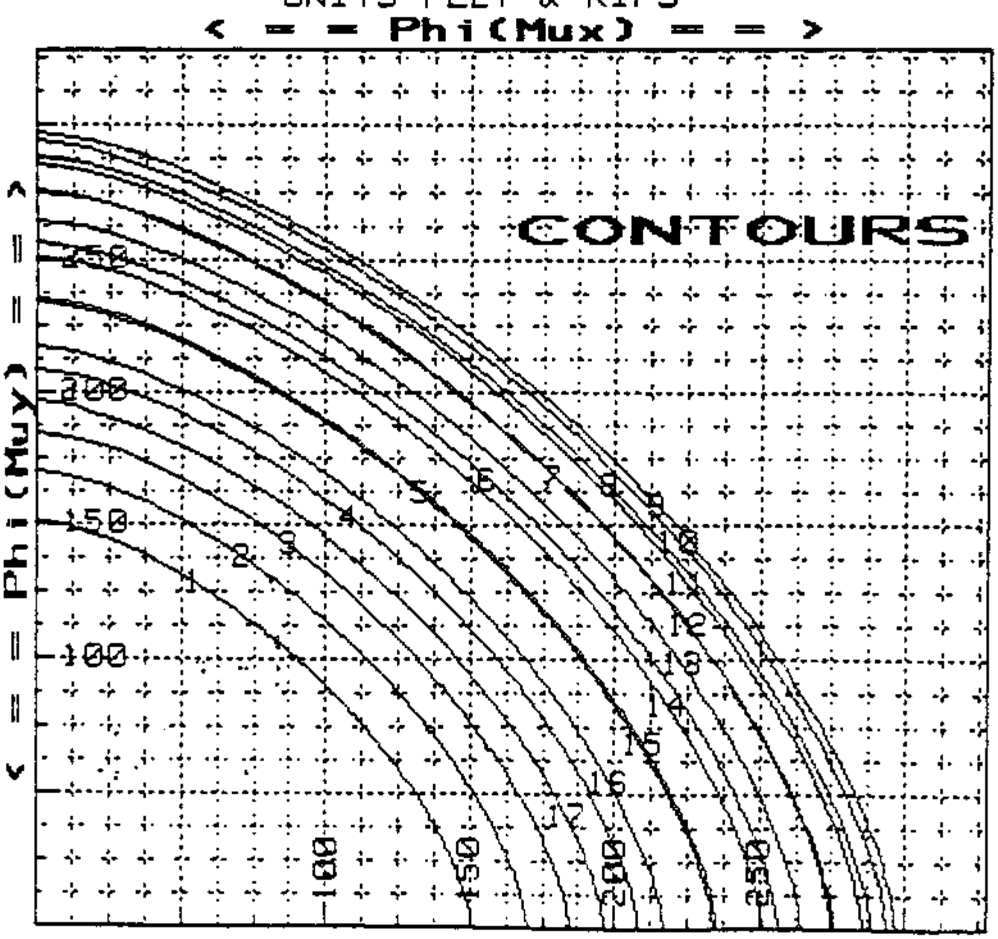
PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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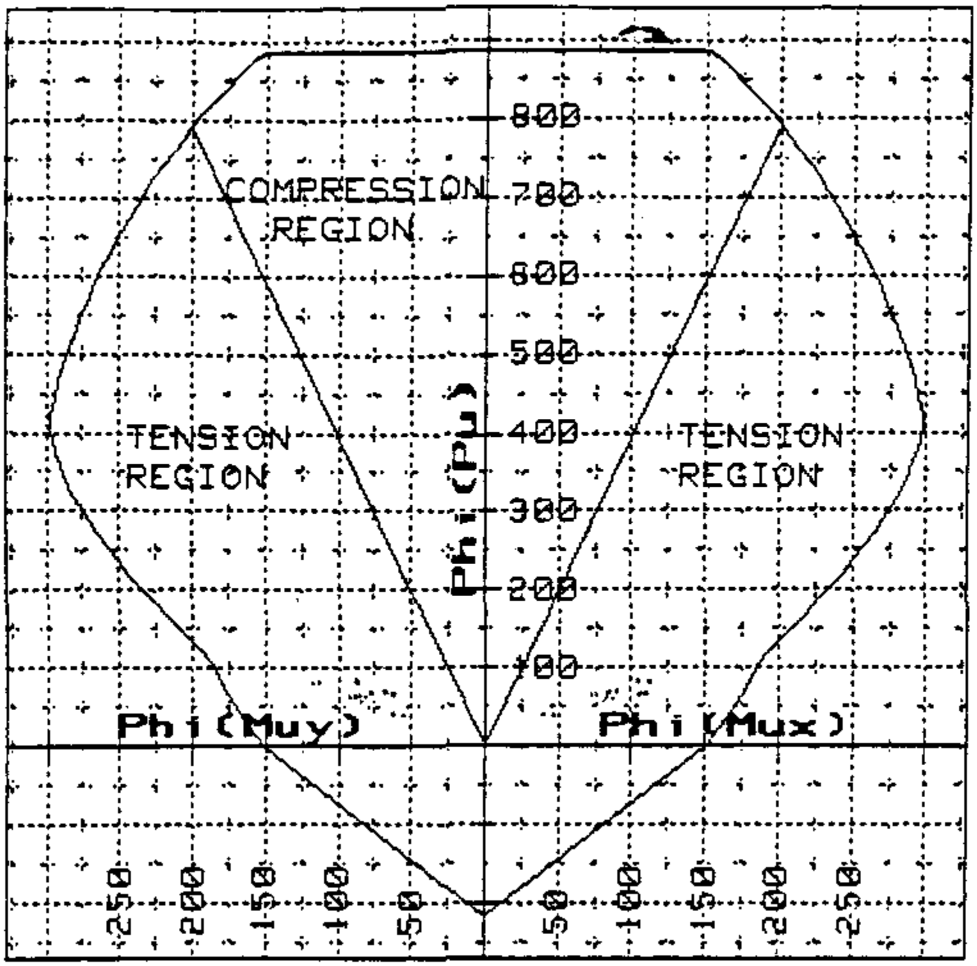

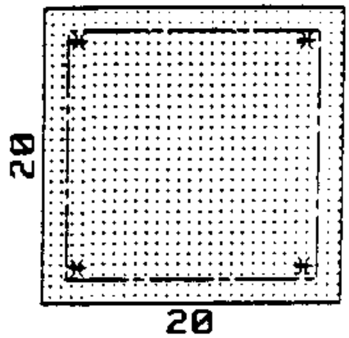

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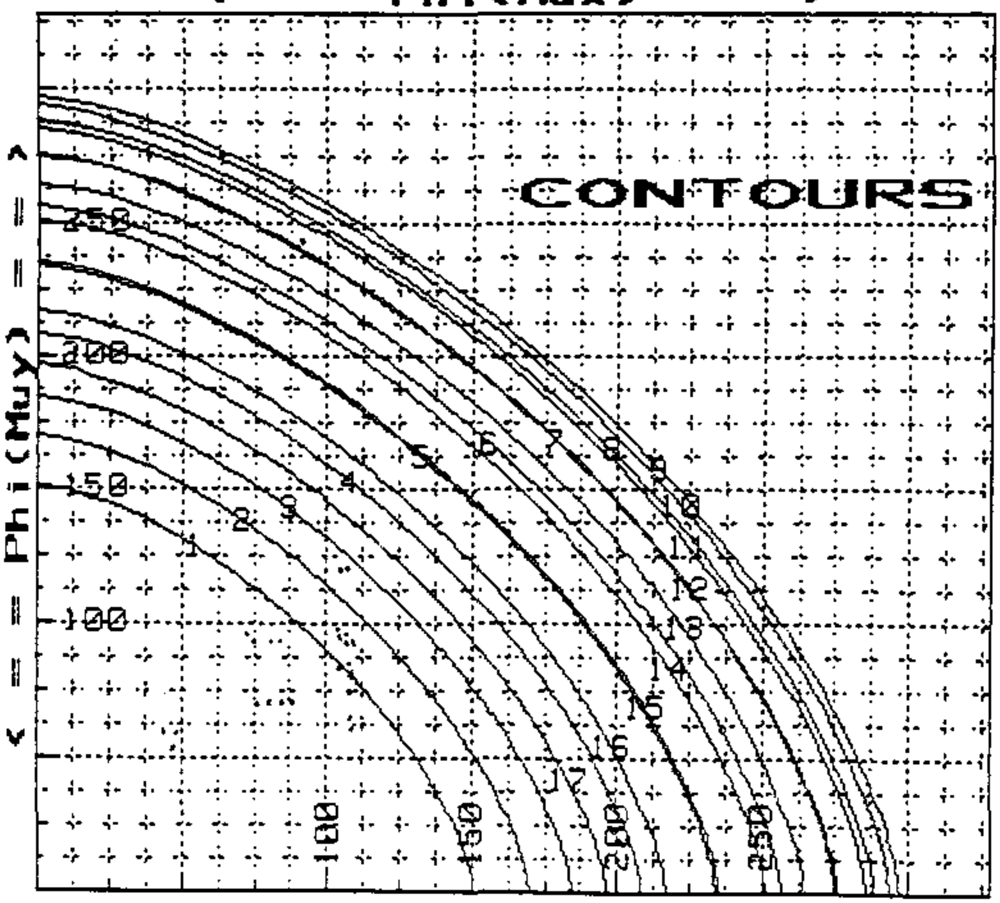

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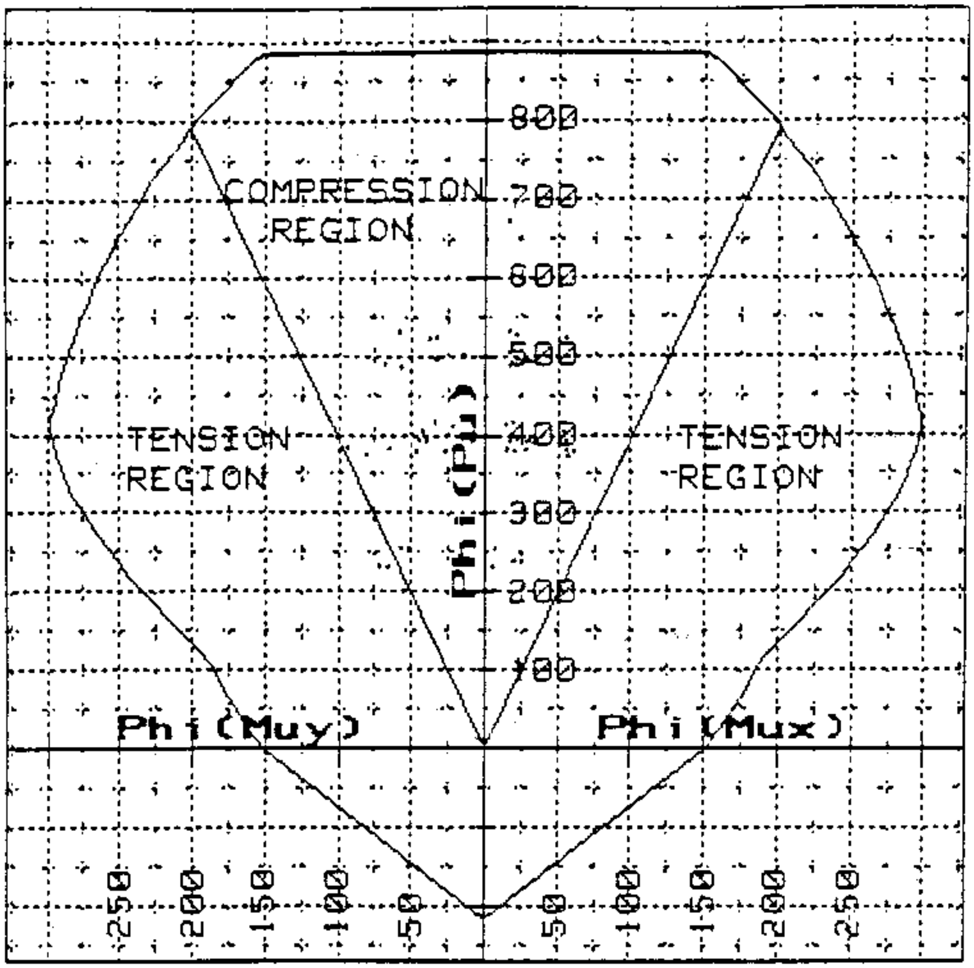

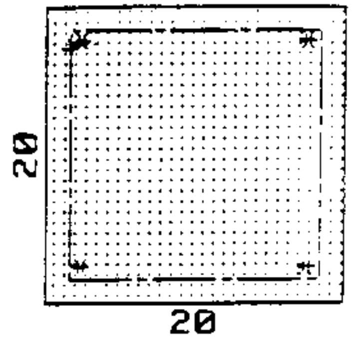

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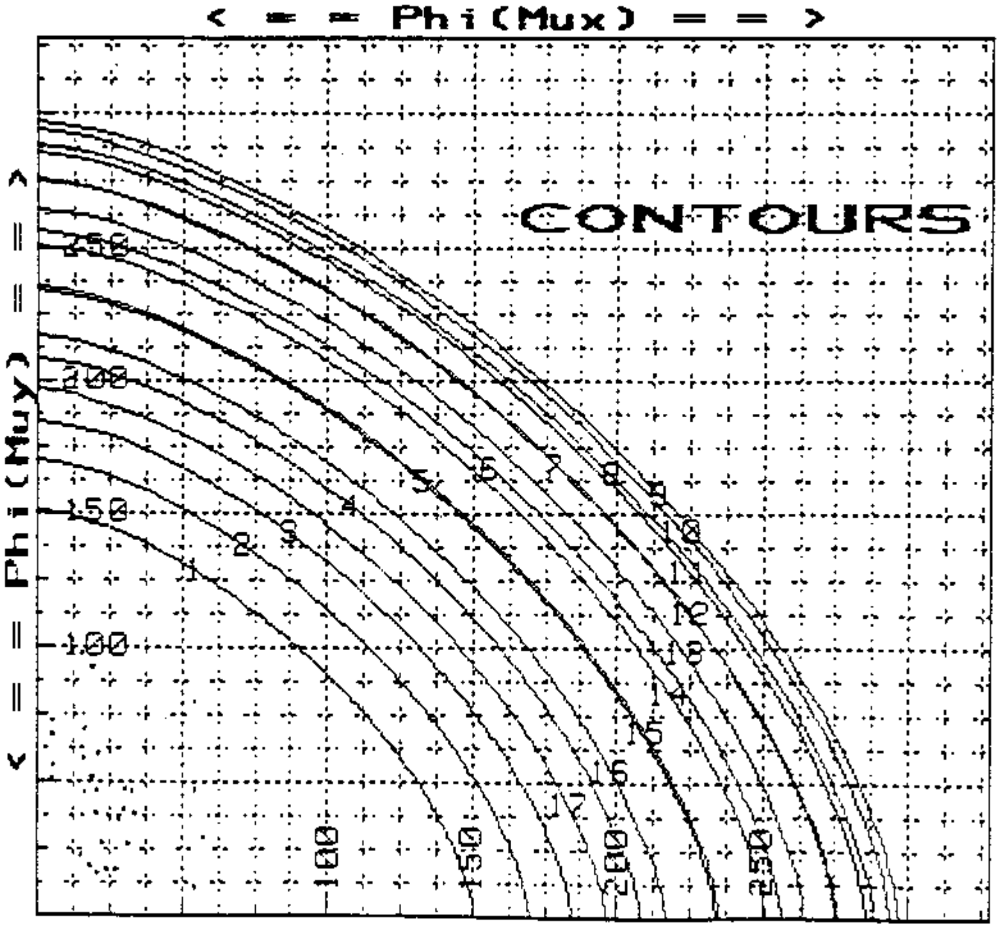
UNITS FEET & KIPS



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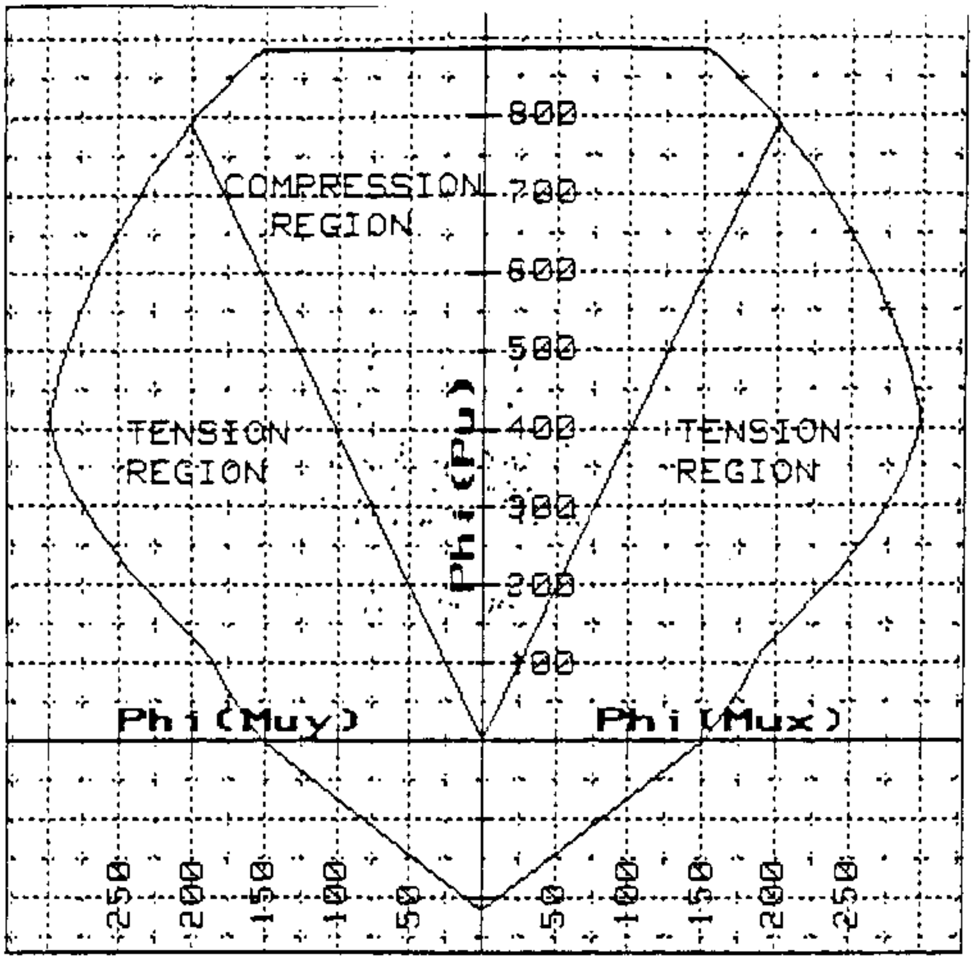

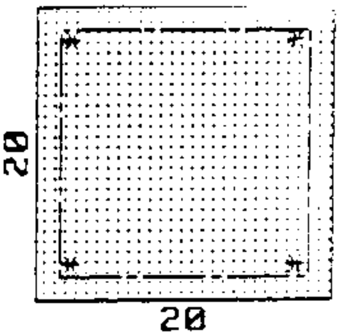

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TOT.BARS= 4
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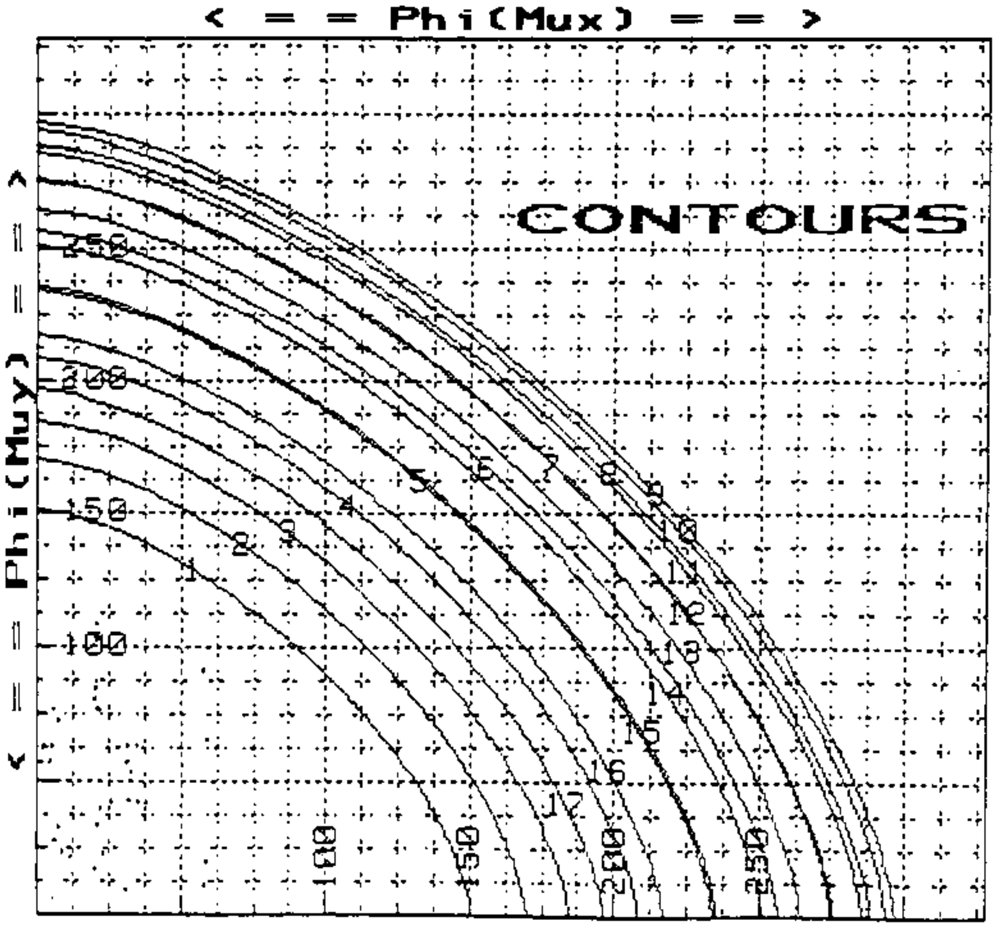
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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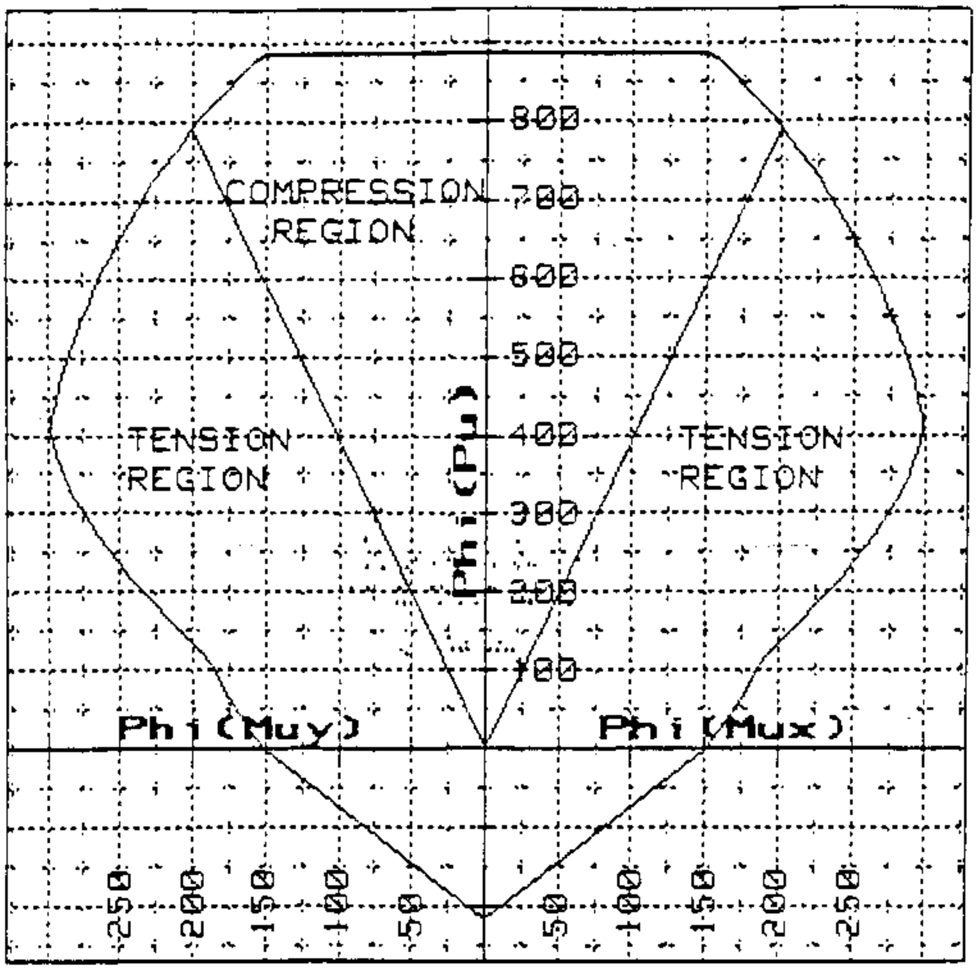

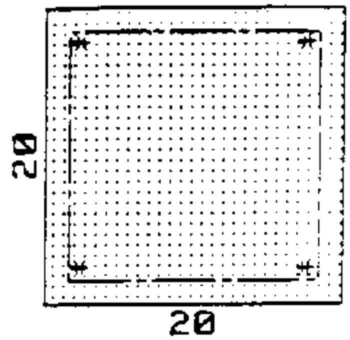

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CORNER BARS
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TOT.BARS= 4
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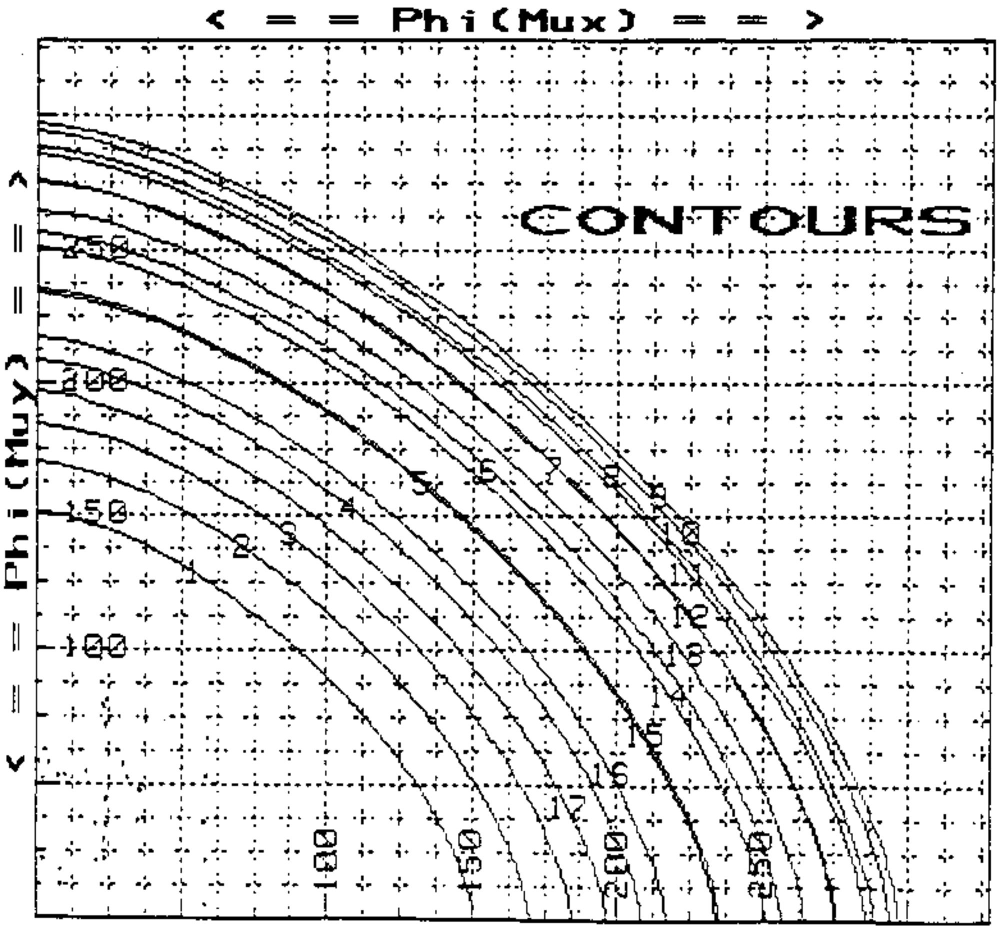
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
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Ix, Iy Steel
Isex= 228.67
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LINE	Pu
1	0
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4	150
5	200
6	250
7	300
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12	550
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14	650
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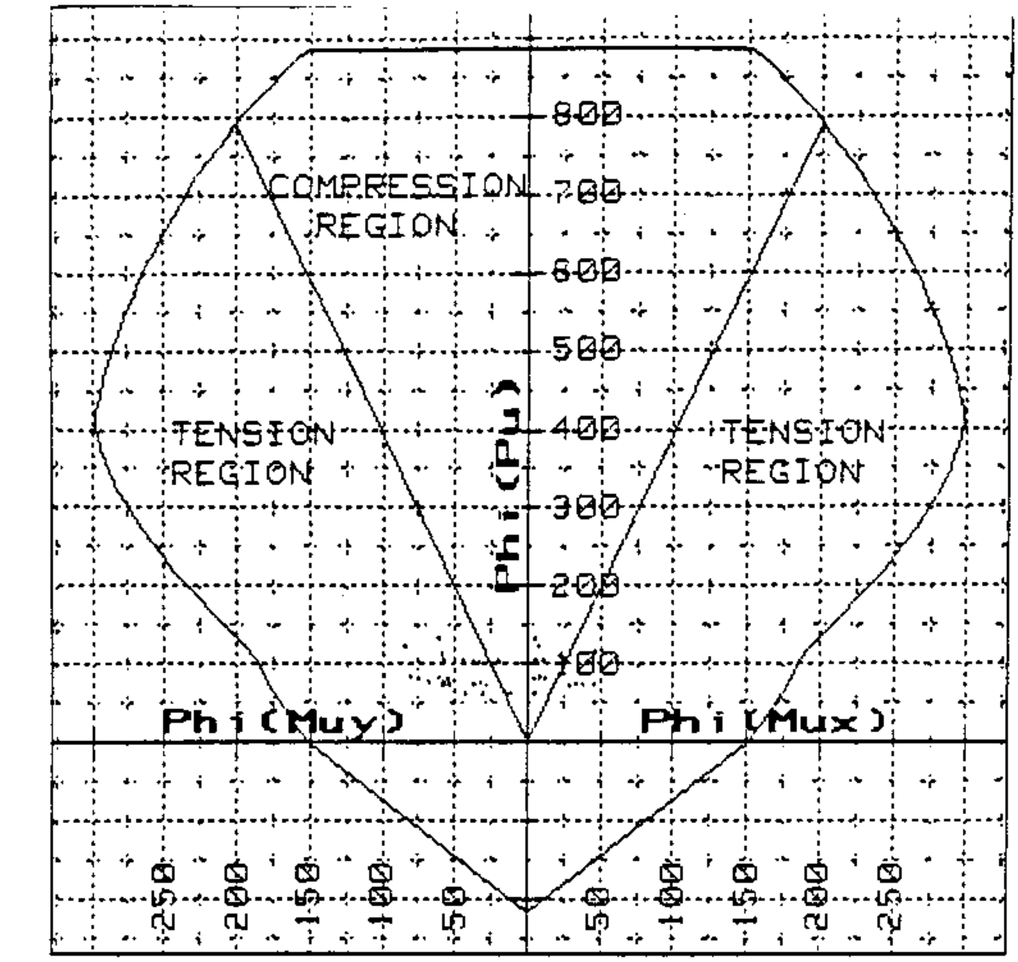
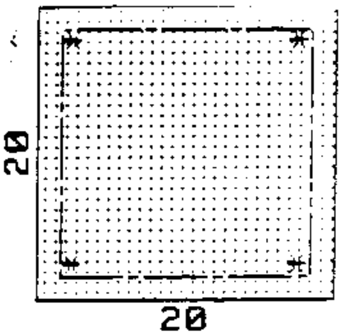
16	750
17	800

B2-3, C2-3



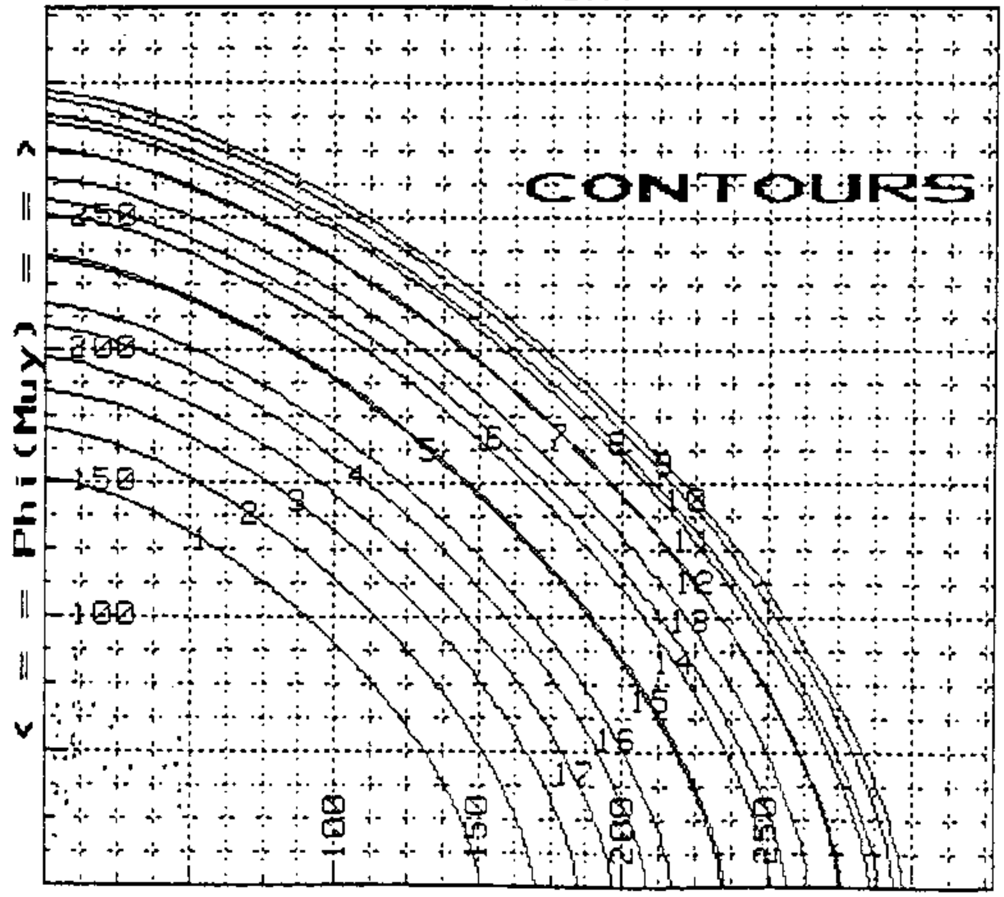


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
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Ix, Iy Steel
Isex= 228.67
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LINE	Pu
1	0
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5	200
6	250
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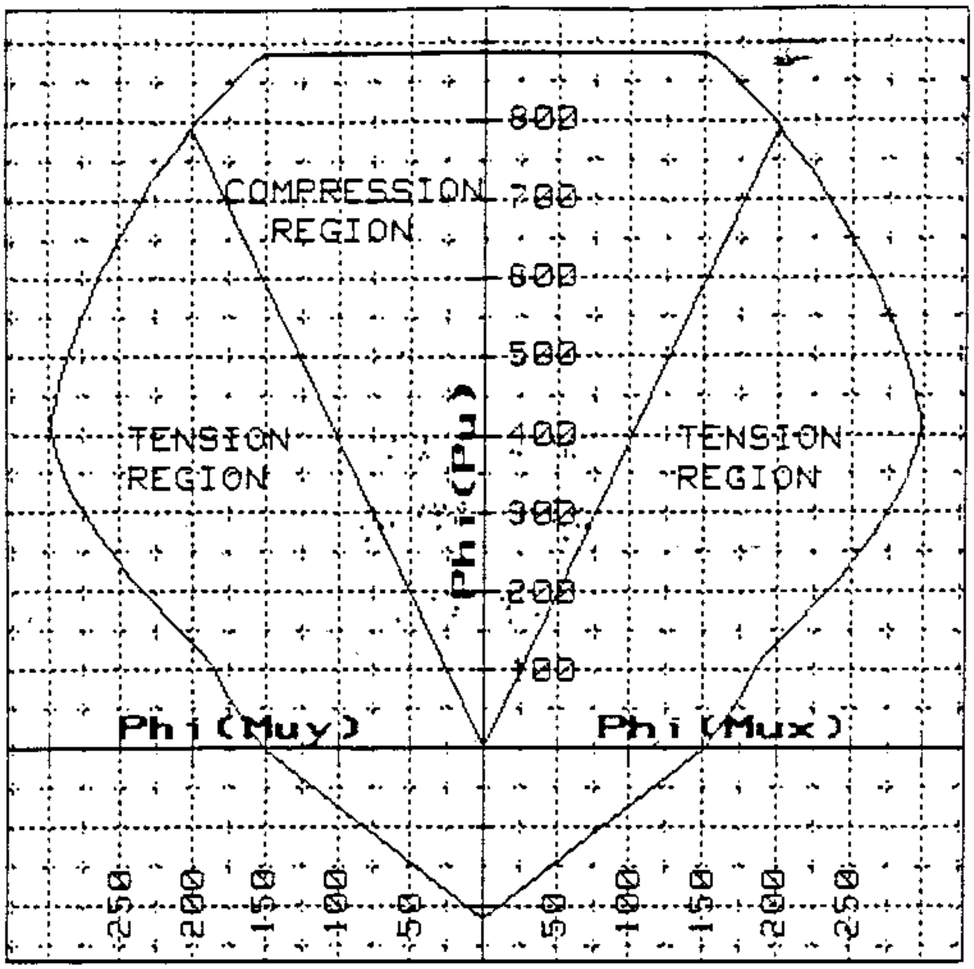
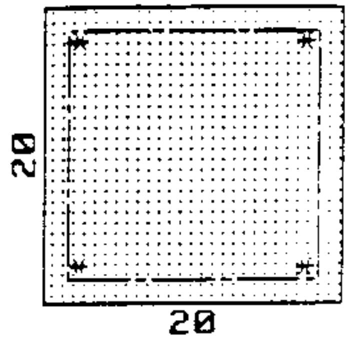
12	550
13	600
14	650
15	700
16	750
17	800

PC-4, C2-4



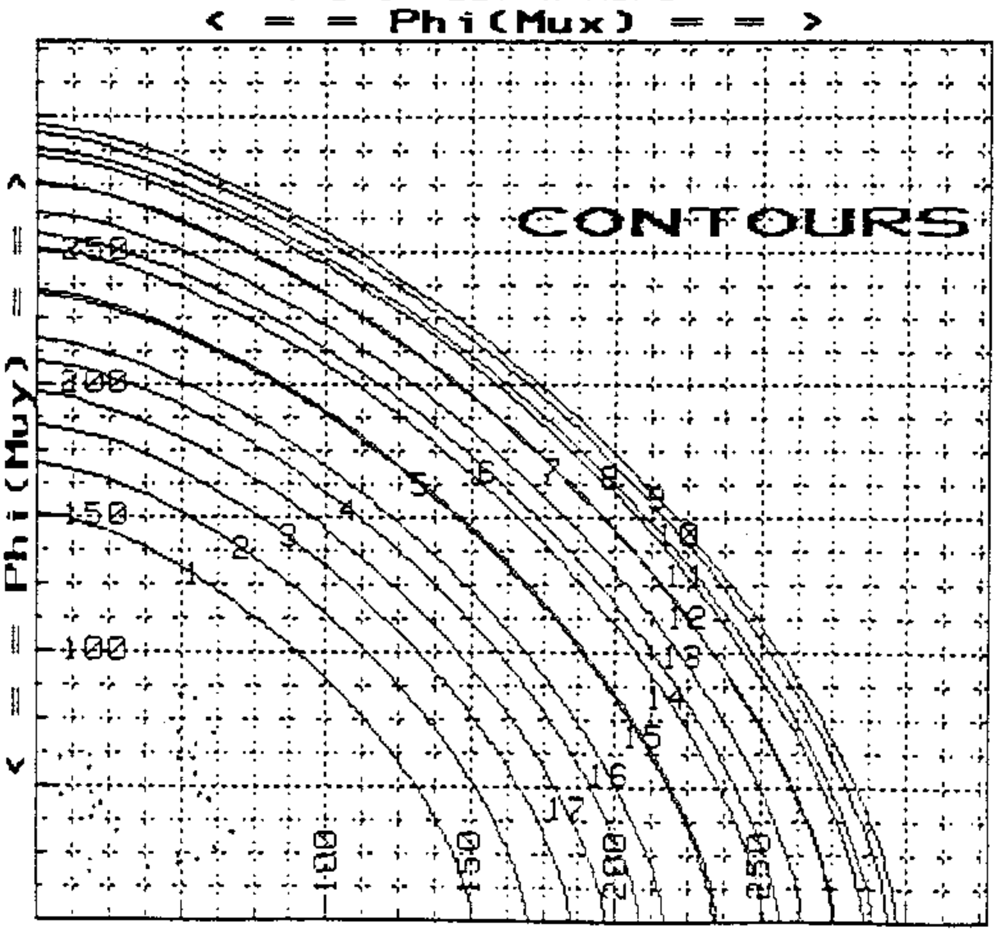


f´c= 4 KSI
Fy = 60 KSI
# 3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
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Ix, Iy Steel
Isex= 228.67
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LINE	Pu
1	0
2	50
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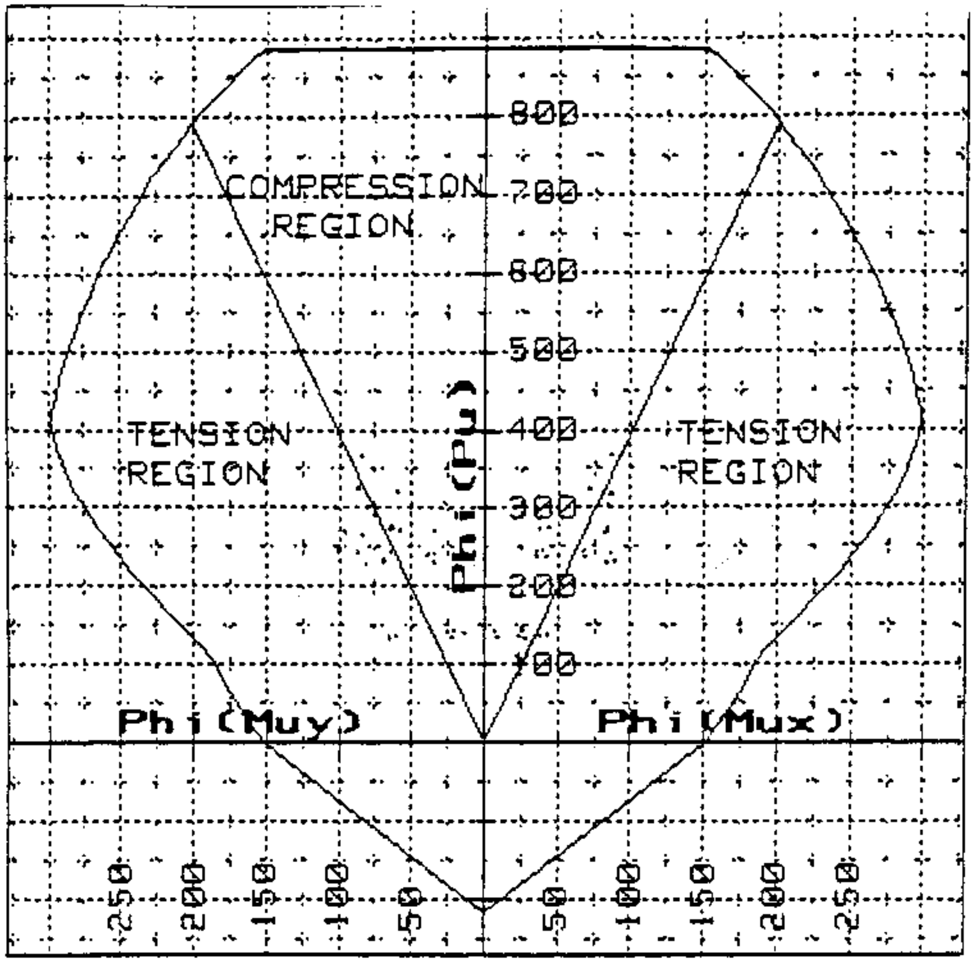

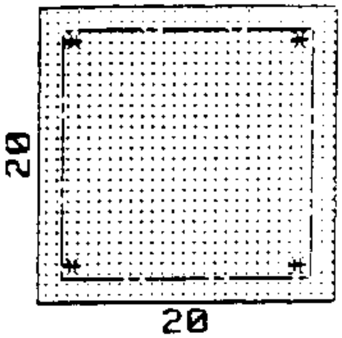

f'c= 4 KSI
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# 3 TIES
@ 18 O.C.
CORNER BARS
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TOT.BARS= 4
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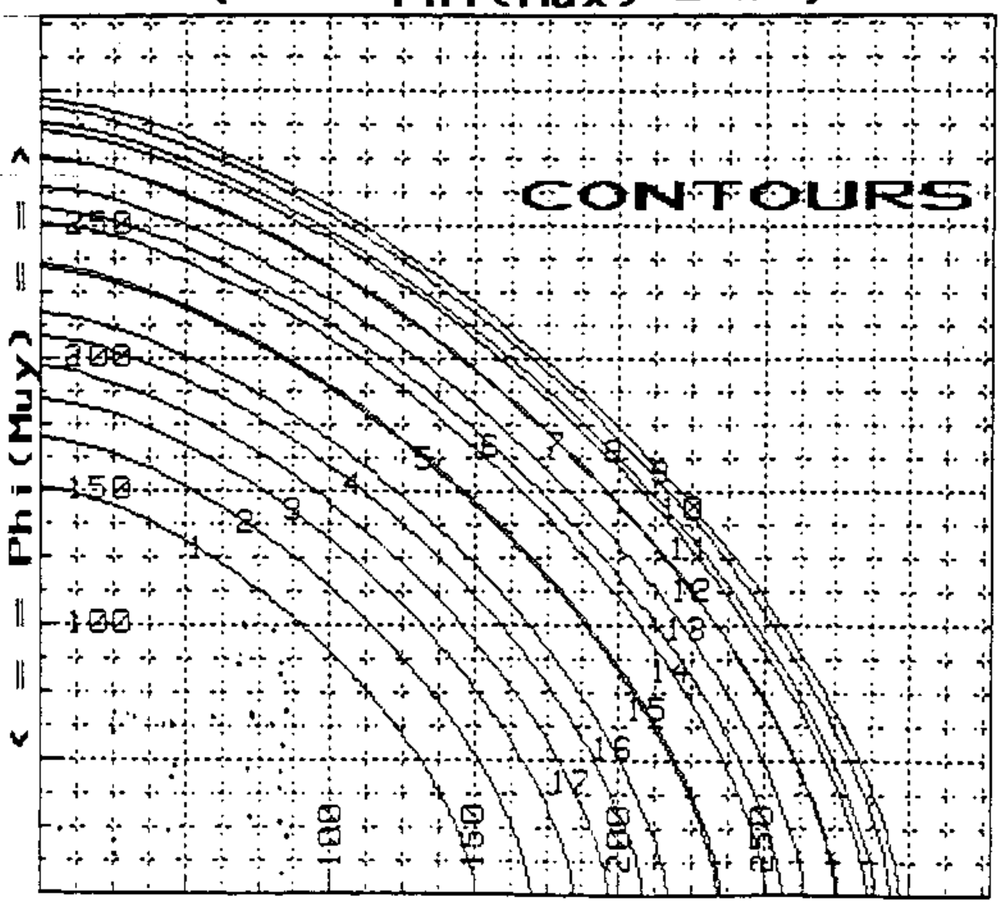
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS

< = = Phi(Mux) = = >



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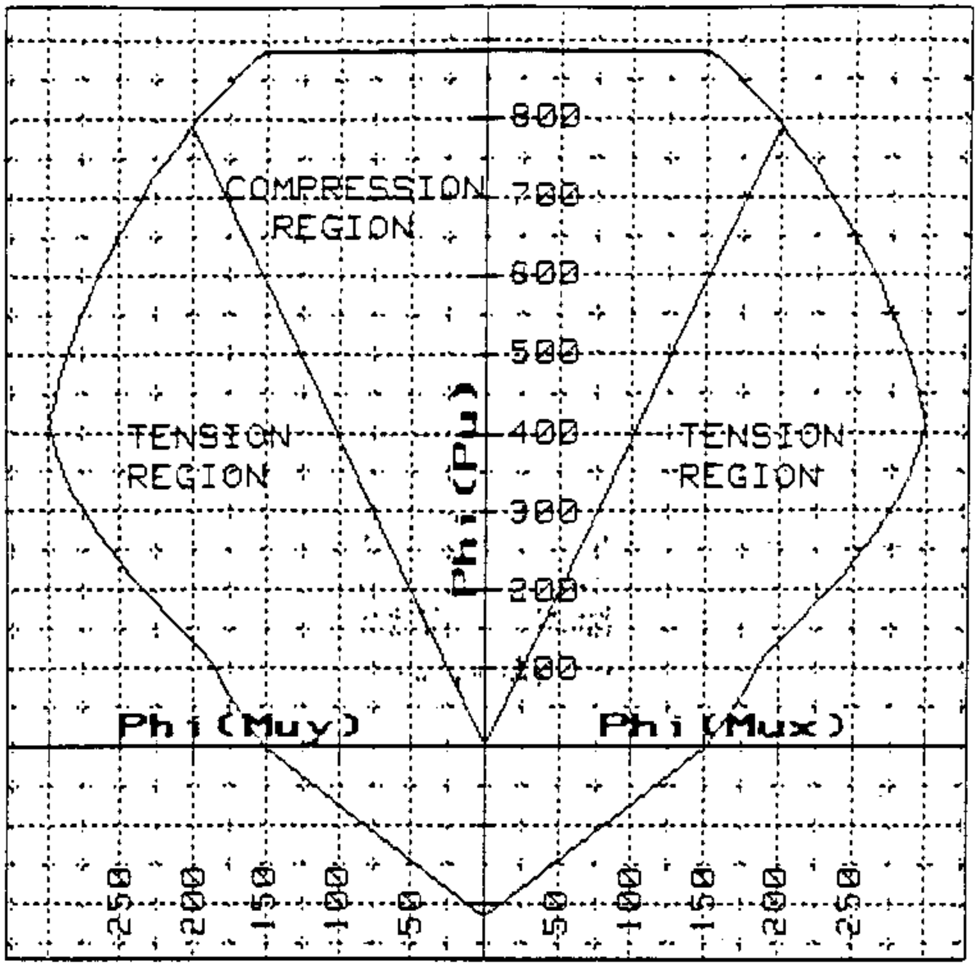
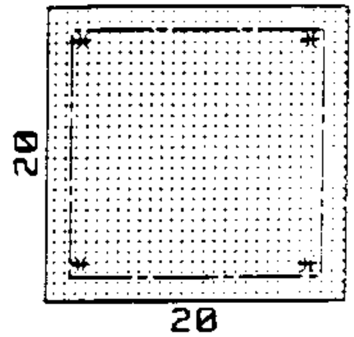
B8-2, C8-2





f´c= 4 KSI
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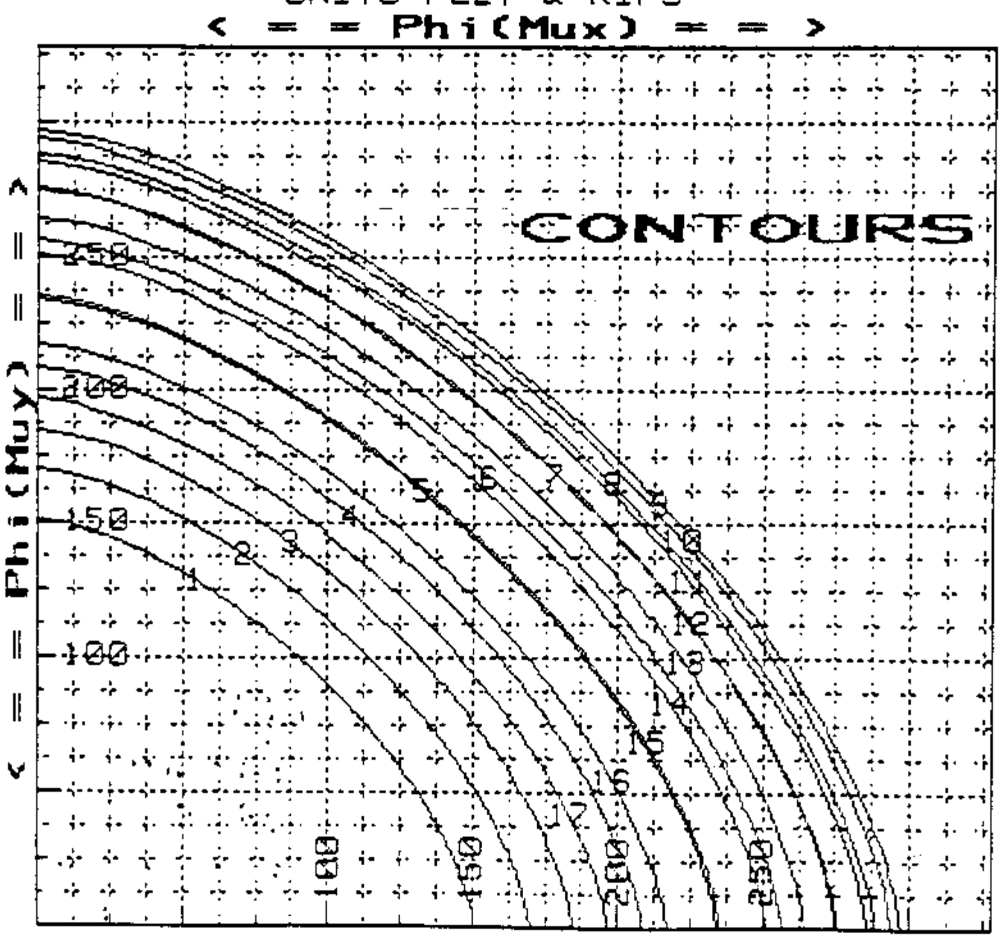
PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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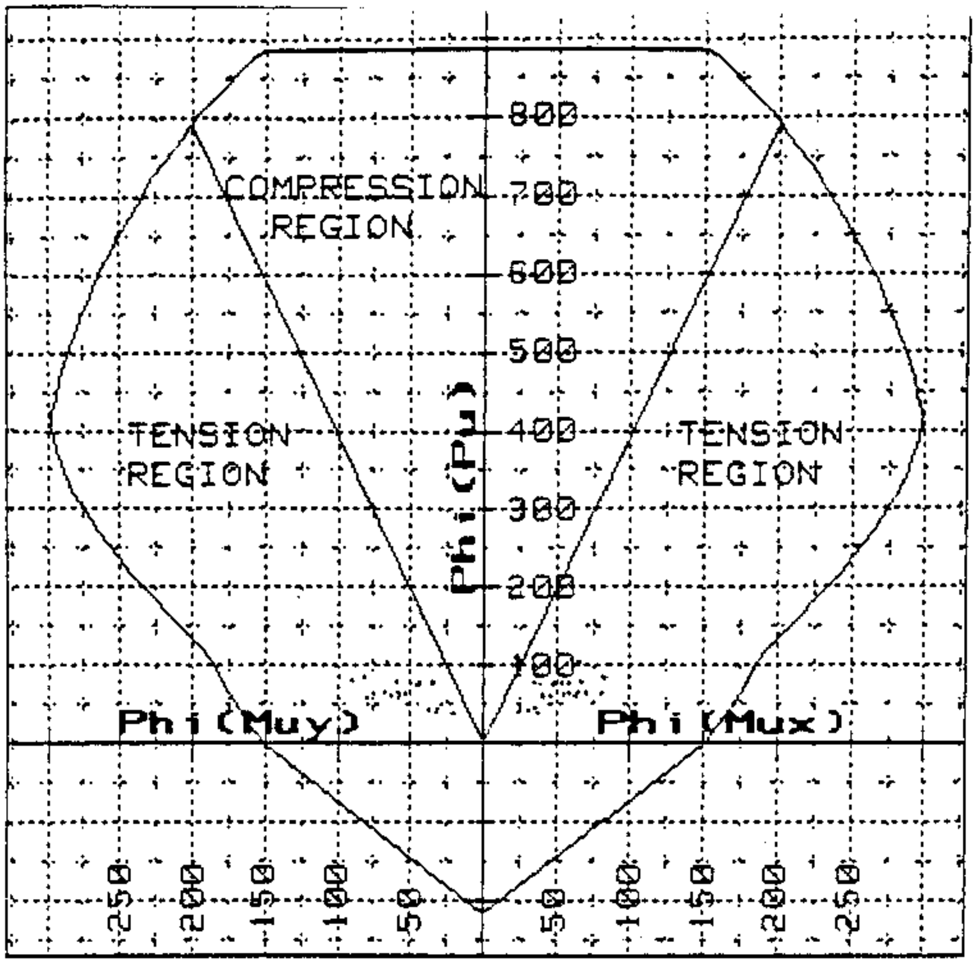

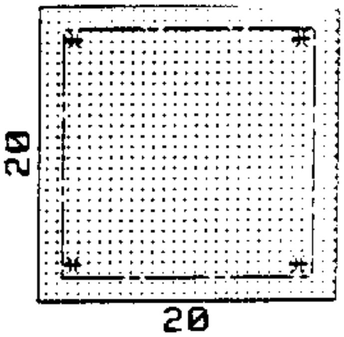

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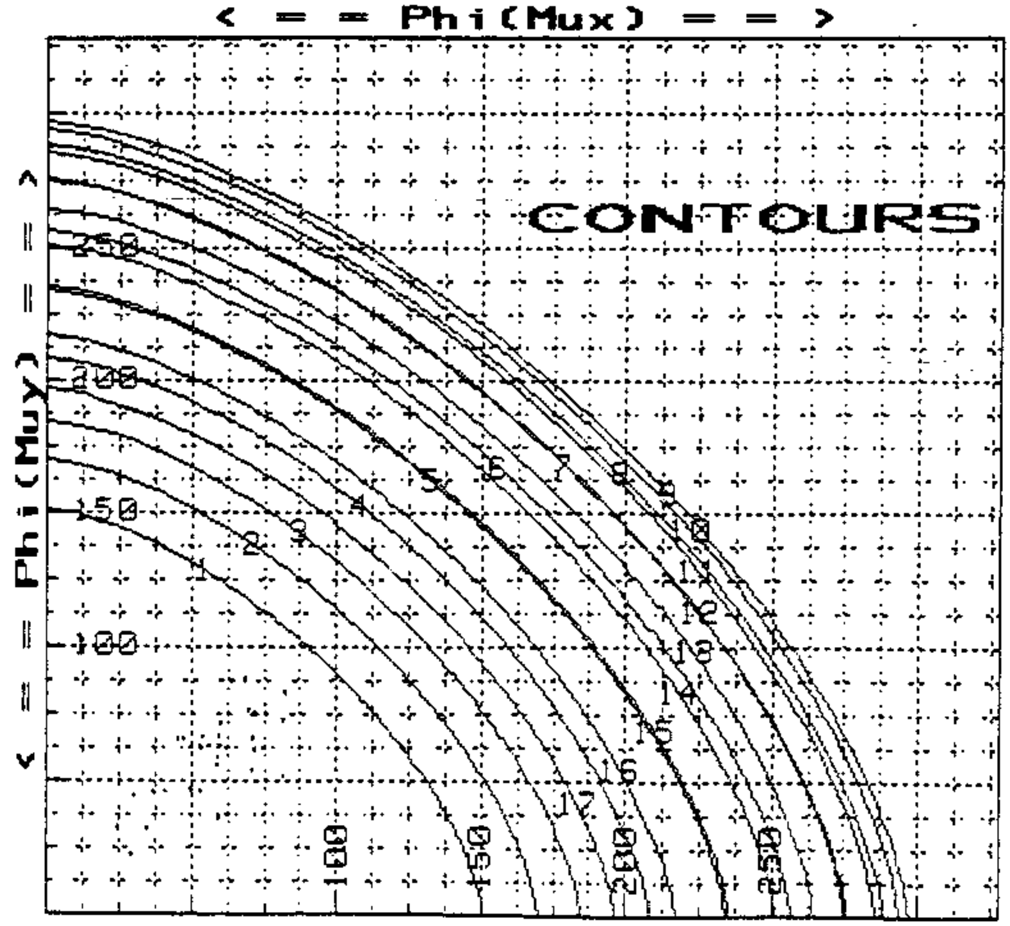
PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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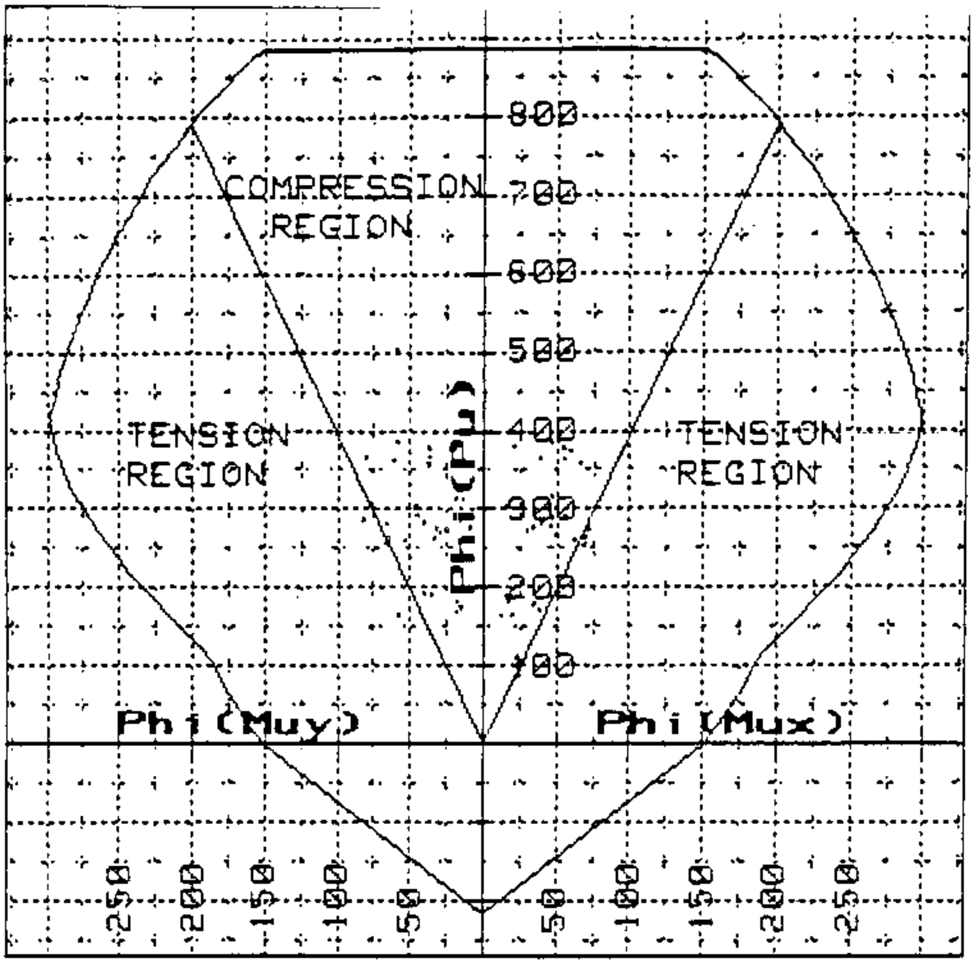
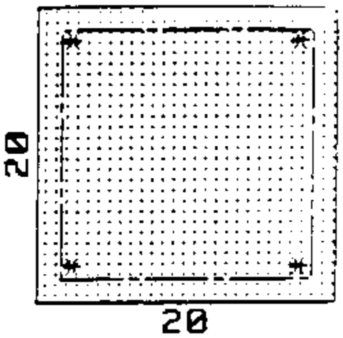
B3-4, C3-4





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PRINCIPAL AXES DIAGRAM

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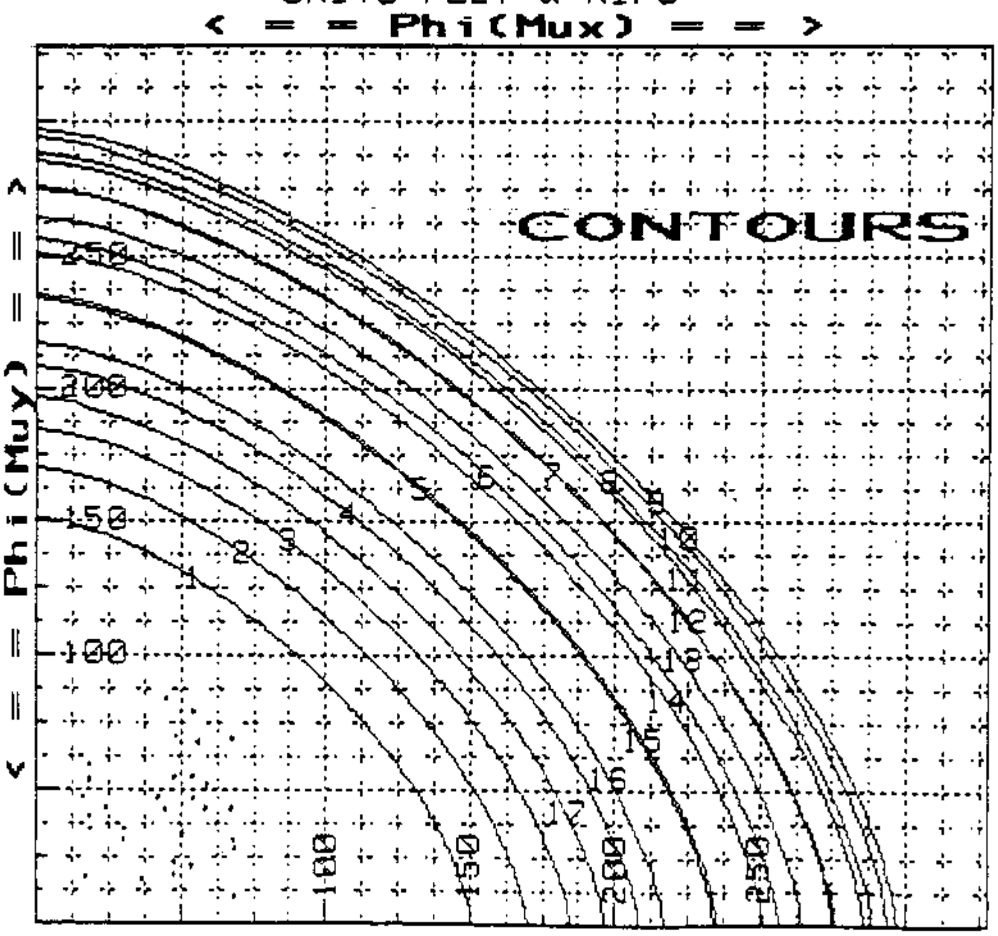
UNITS FEET & KIPS



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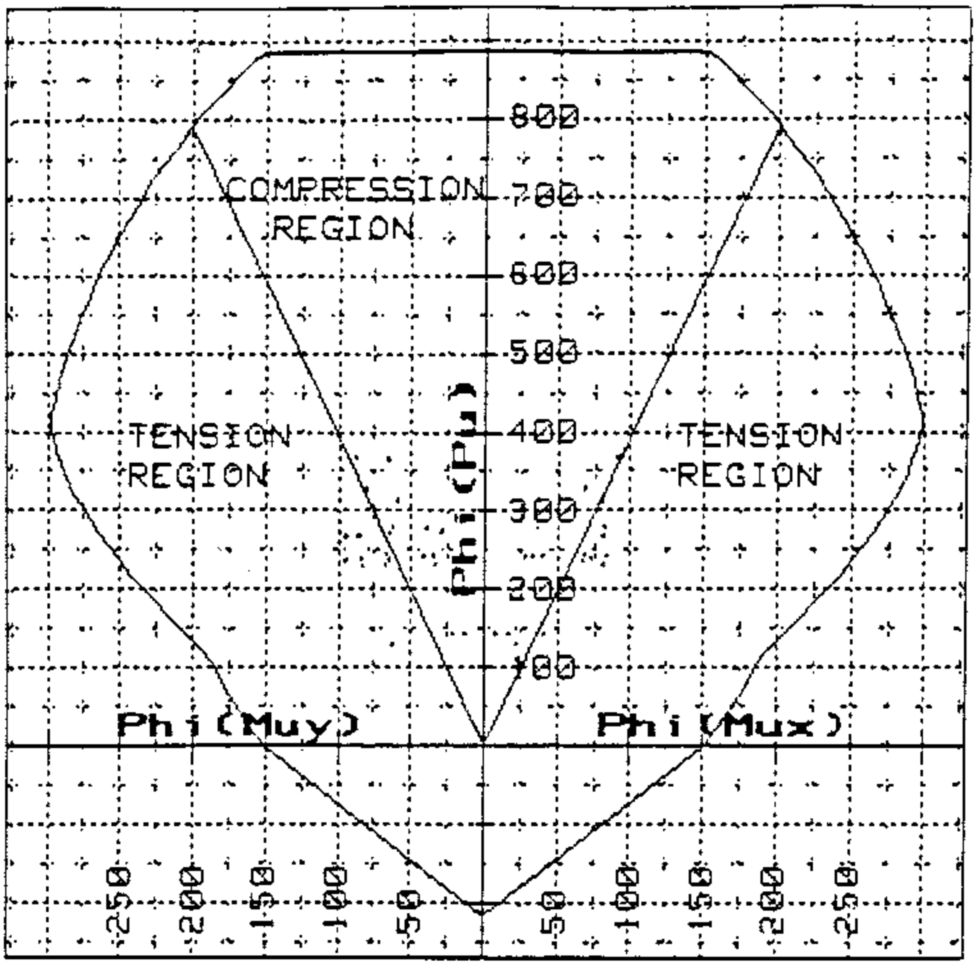

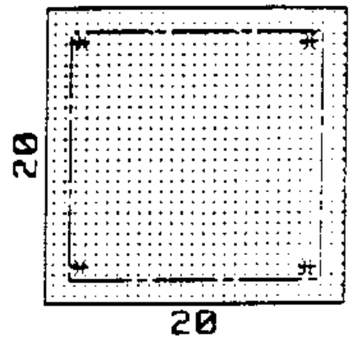

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PRINCIPAL AXES DIAGRAM

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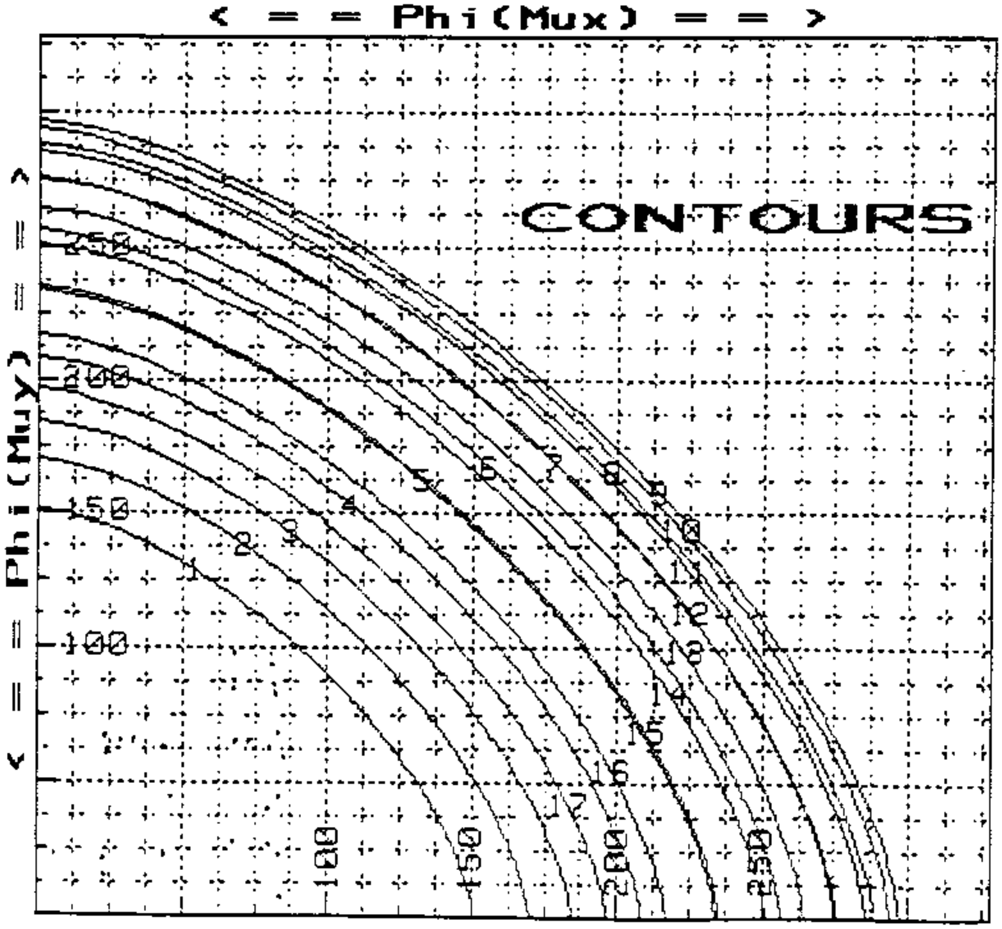
UNITS FEET & KIPS

< = = Phi(Mux) = = >



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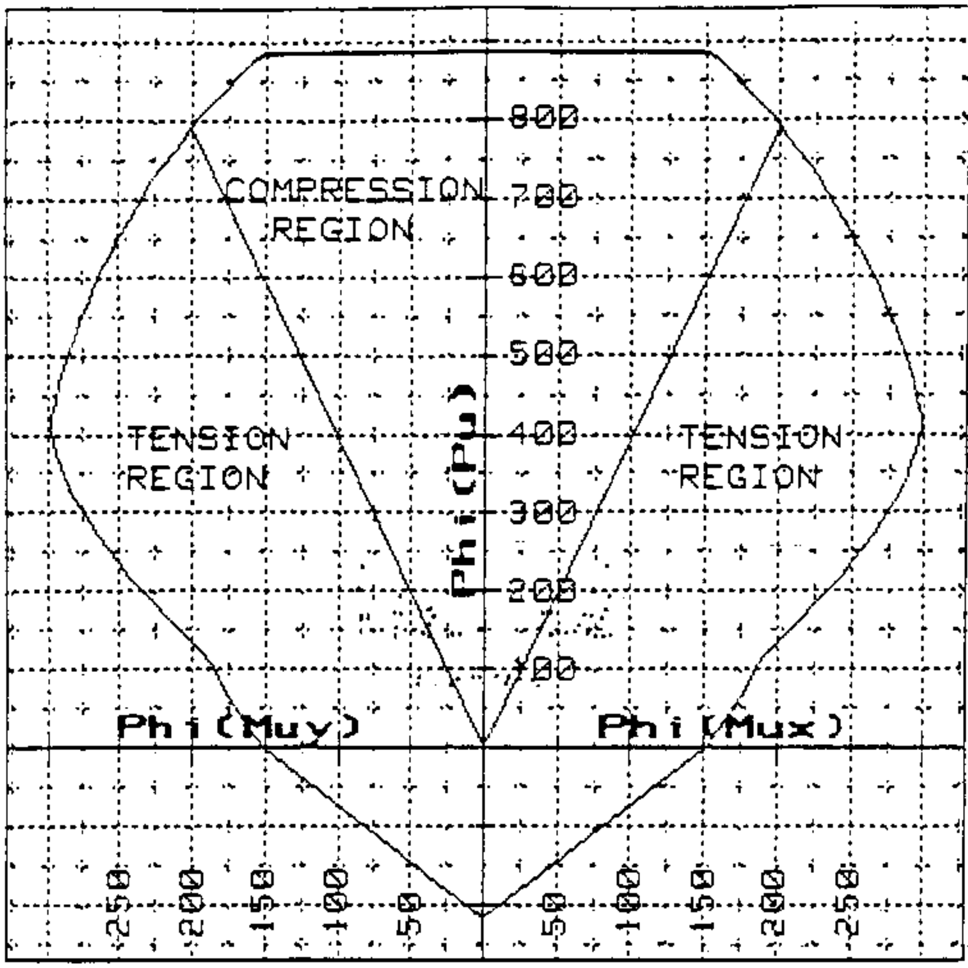
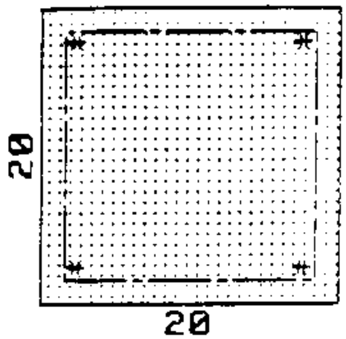
B4-2, C4-2





f´c= 4 KSI
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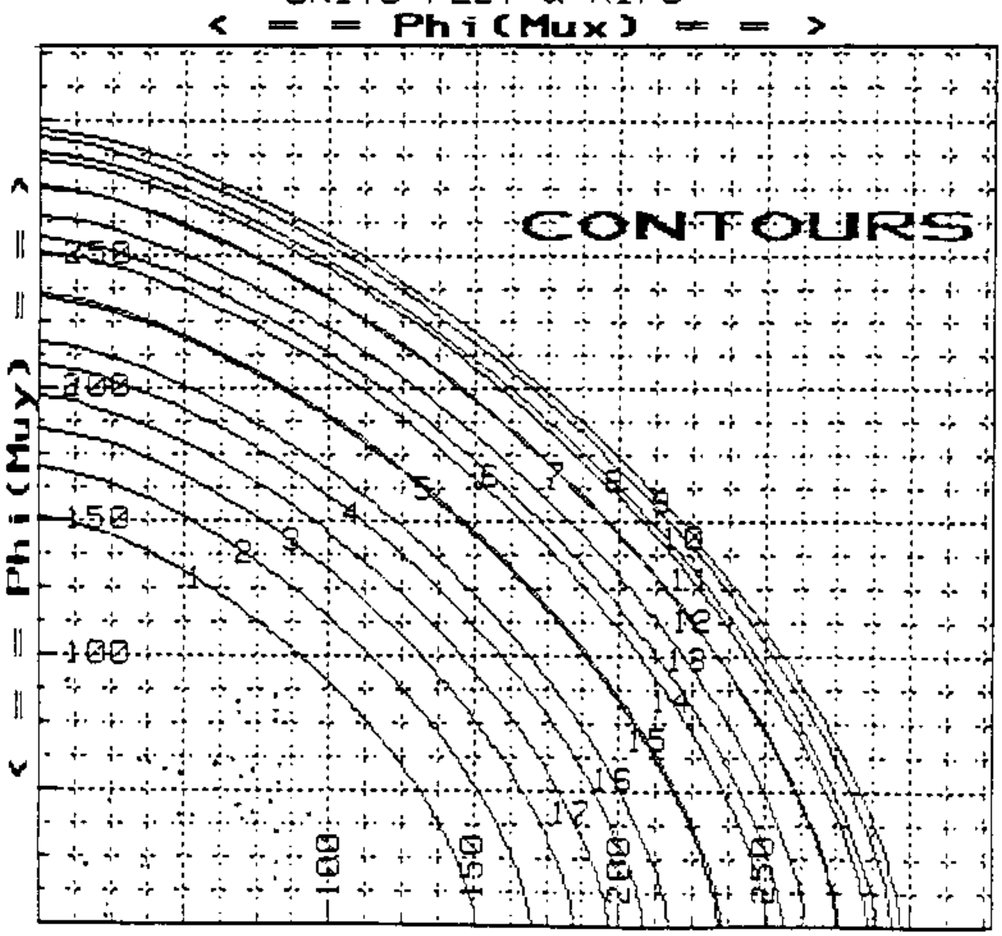
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



Ix, Iy Gross
Ix= 13333.33
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Ix, Iy Steel
Isex= 228.67
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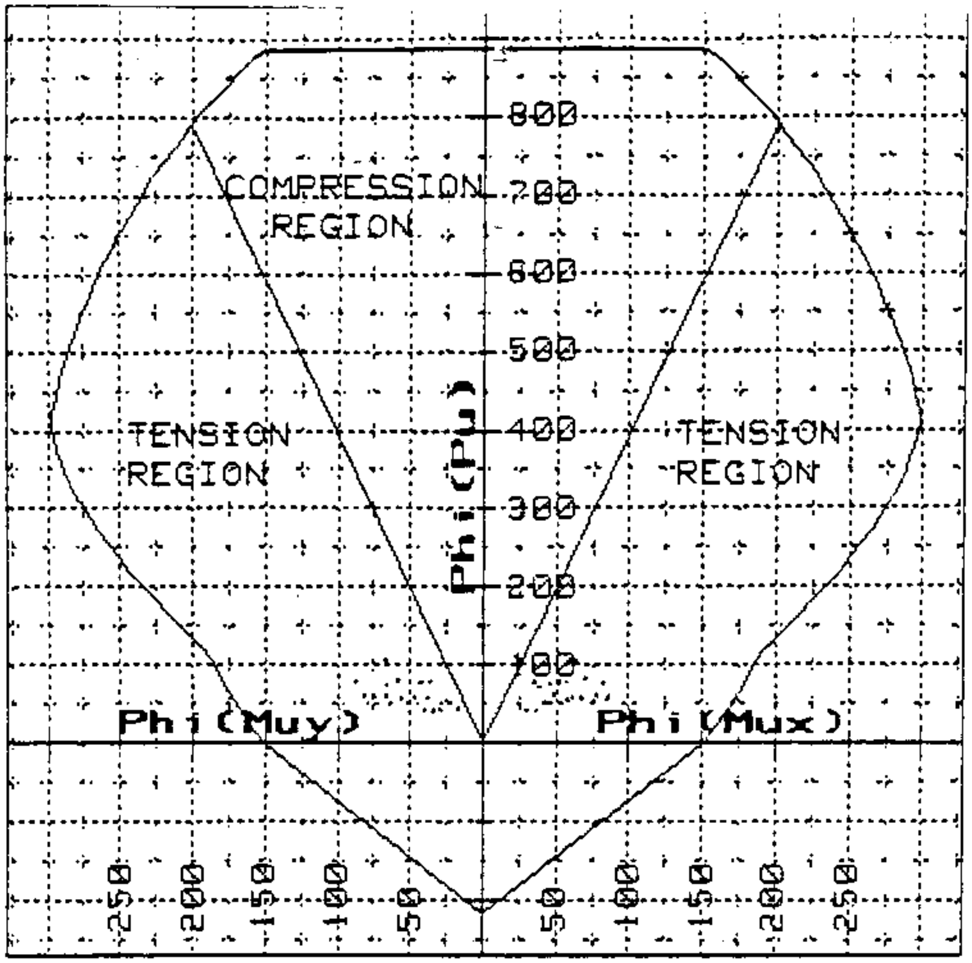

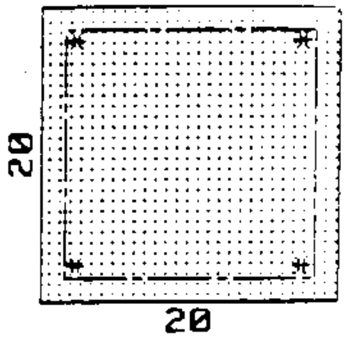

f´c= 4 KSI
~~Fy = 60 KSI~~
3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
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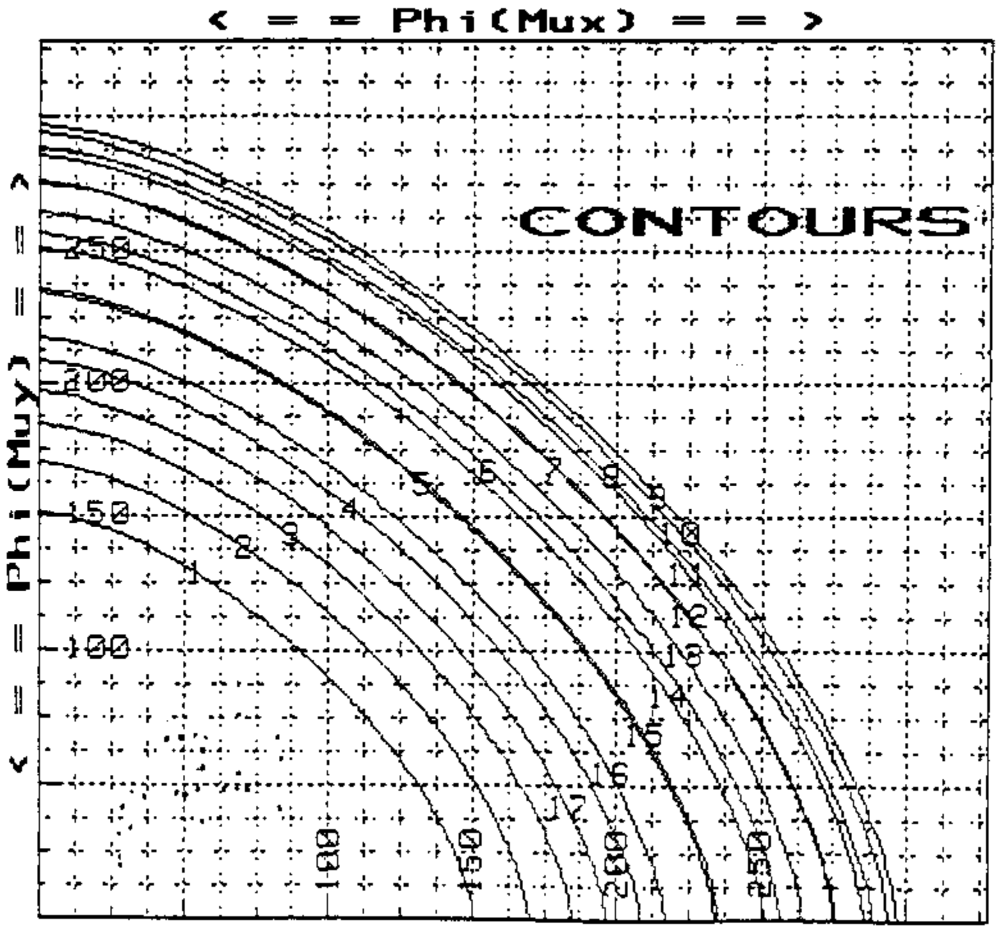

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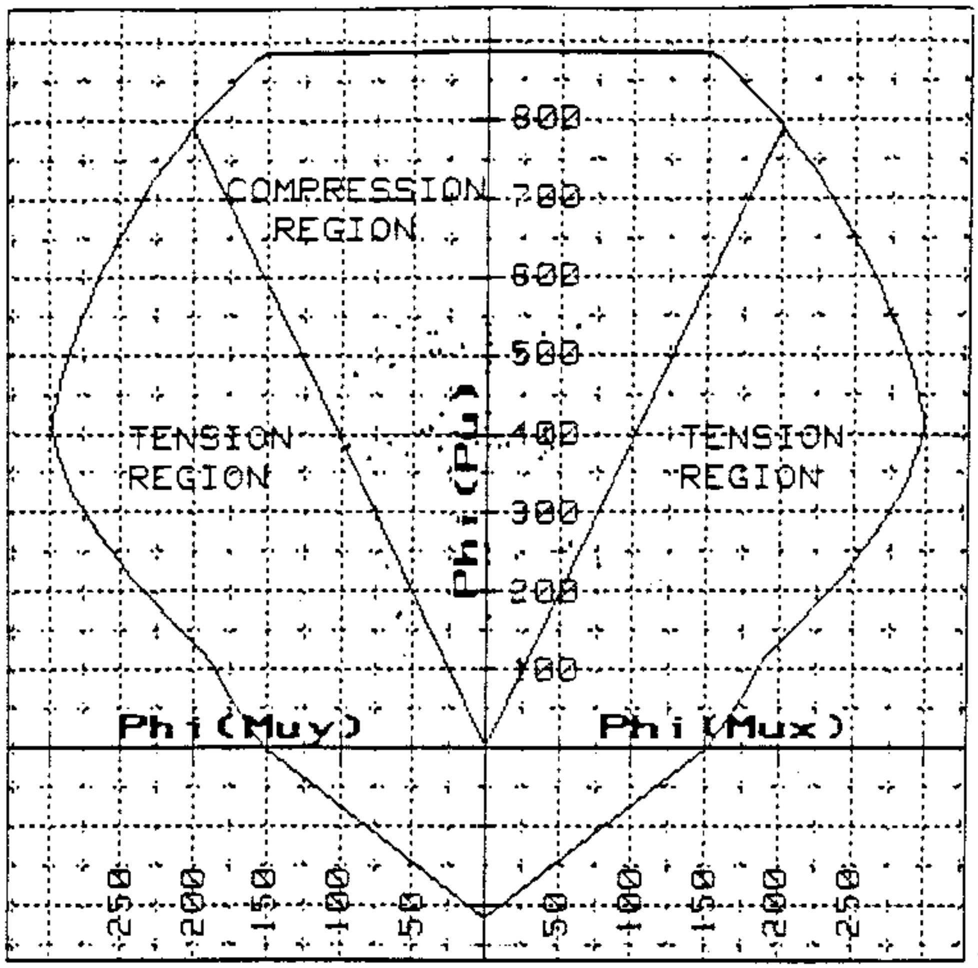
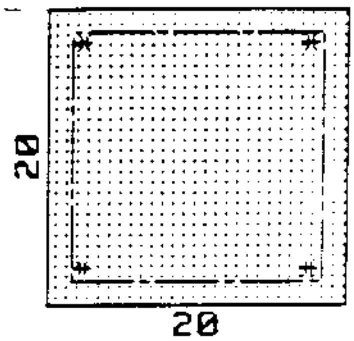
B4-4, C4-4





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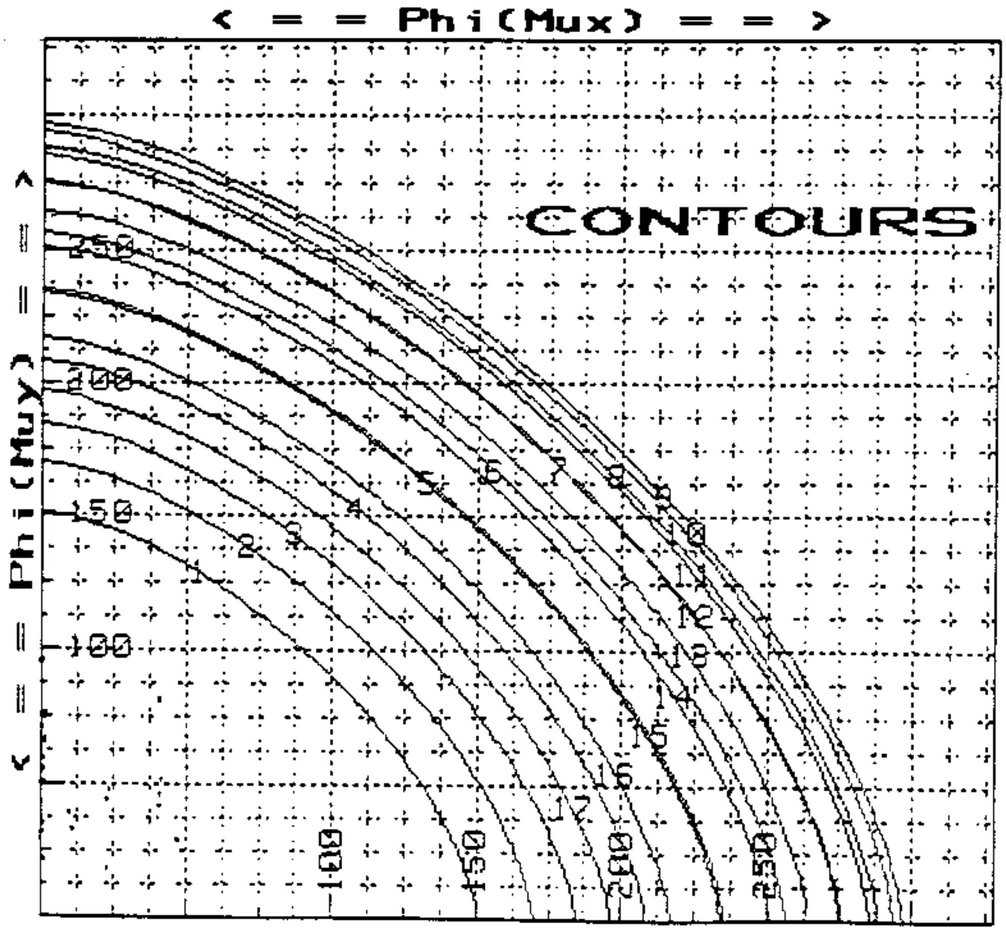

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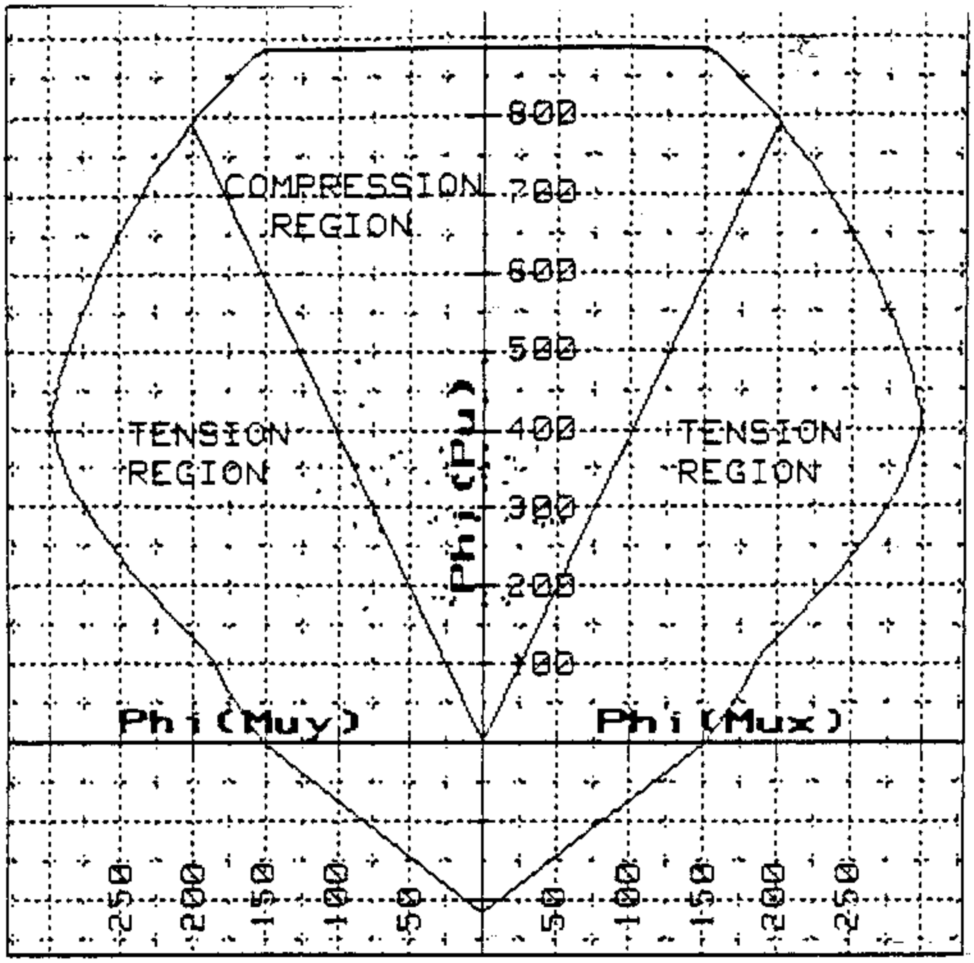

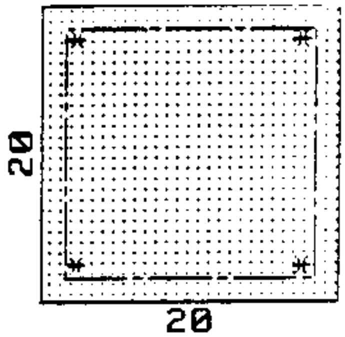

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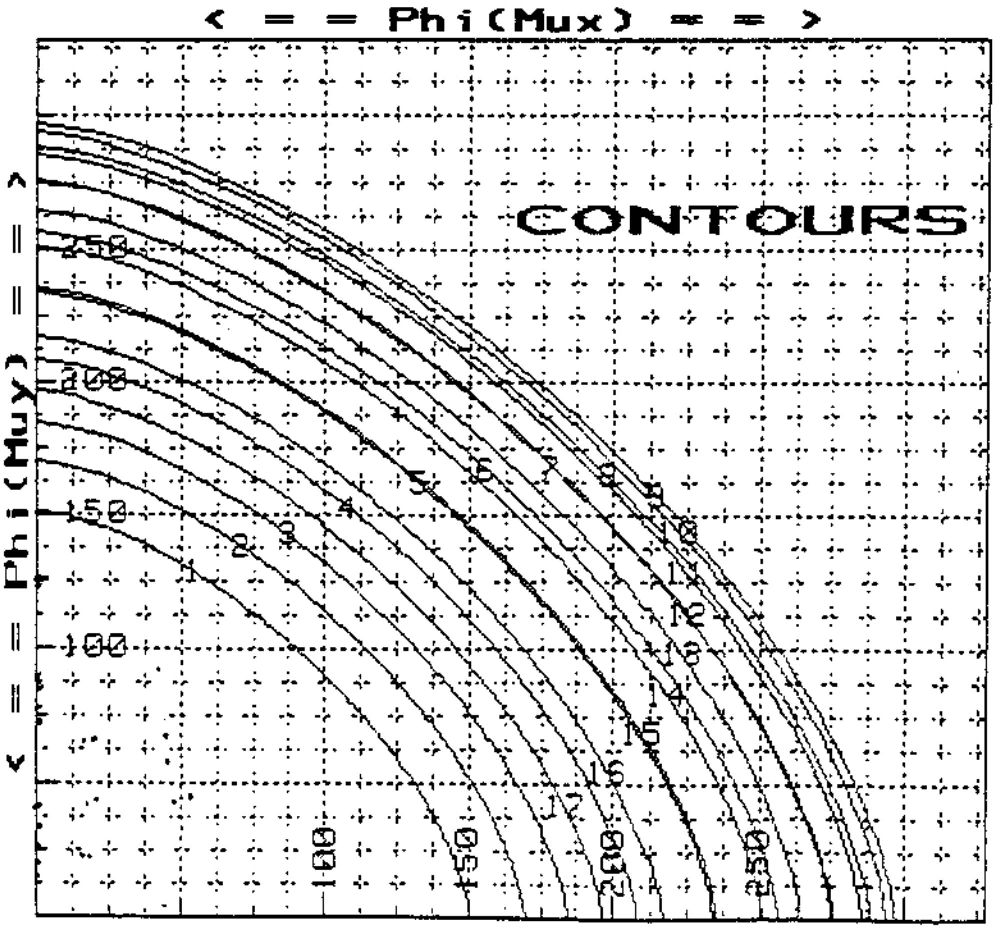

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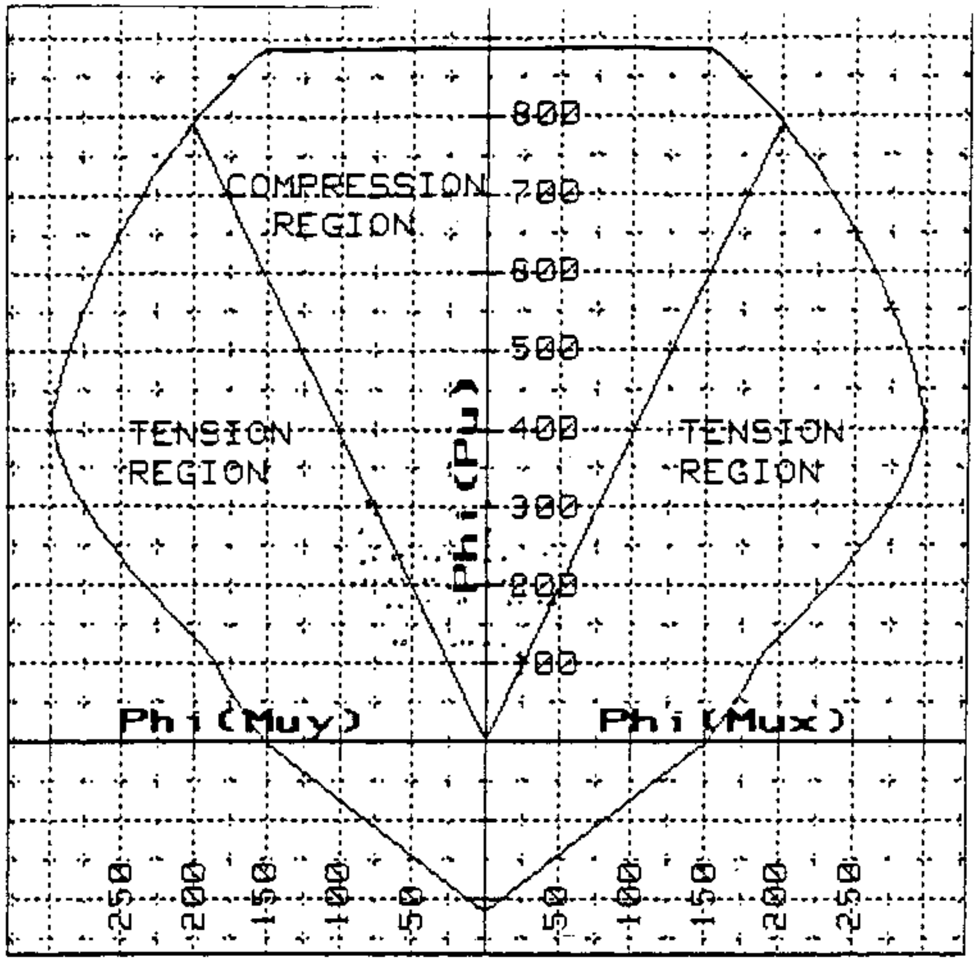

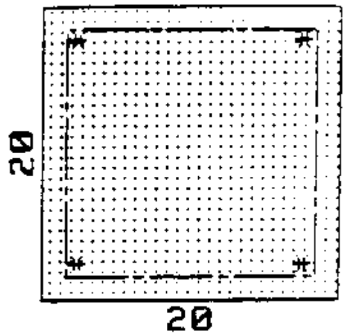

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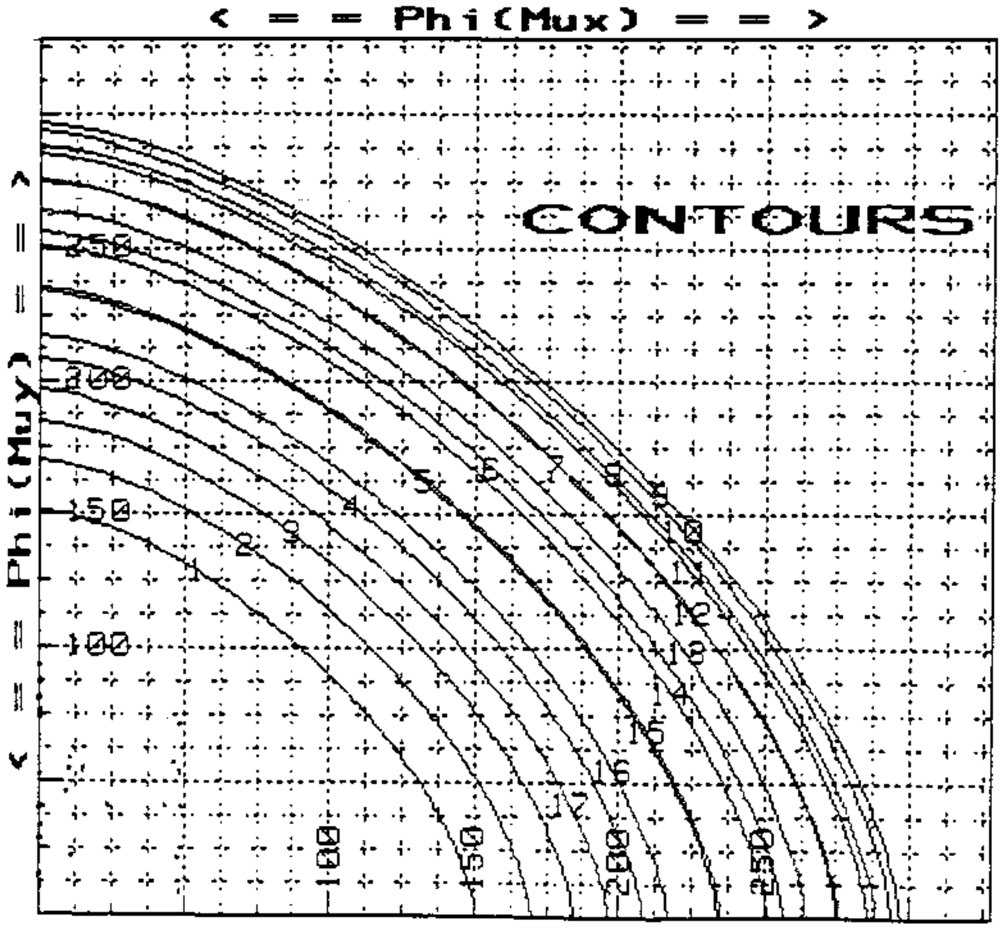

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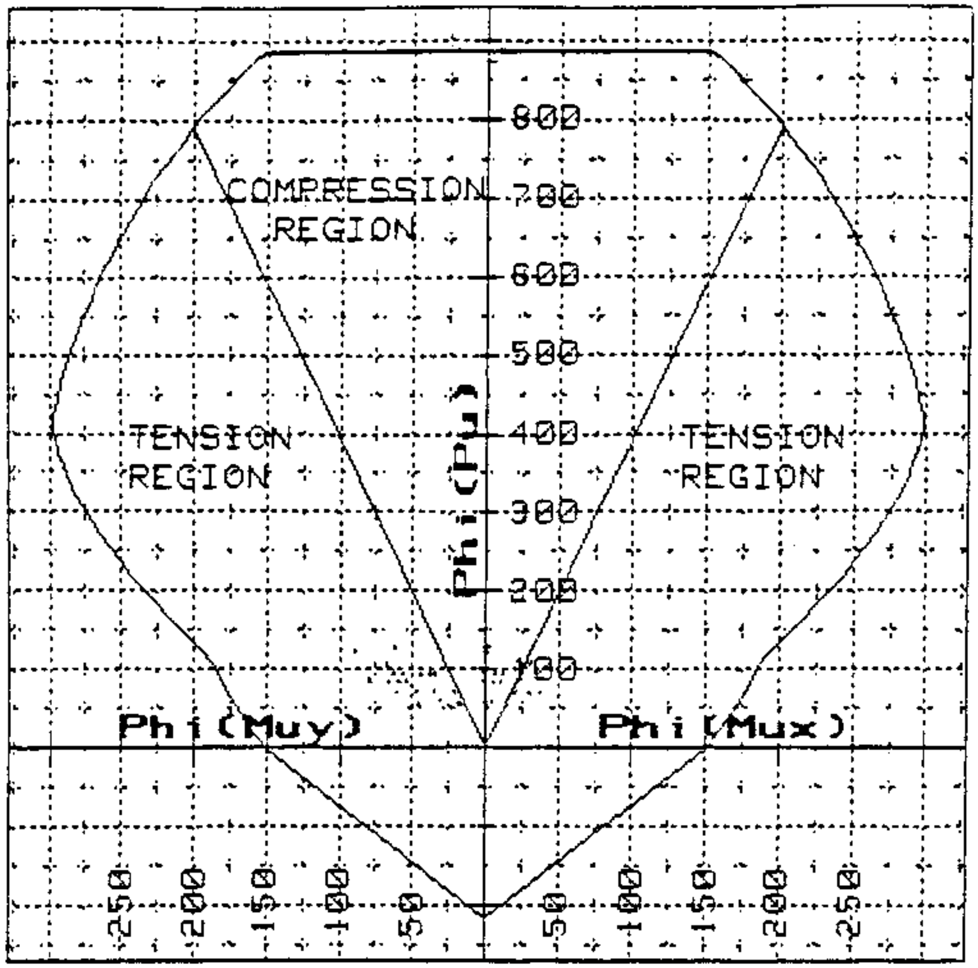

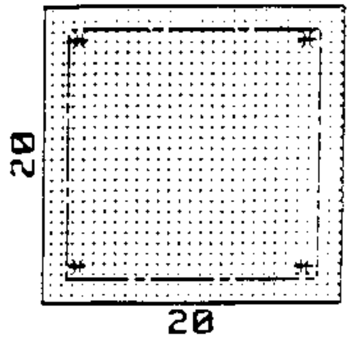

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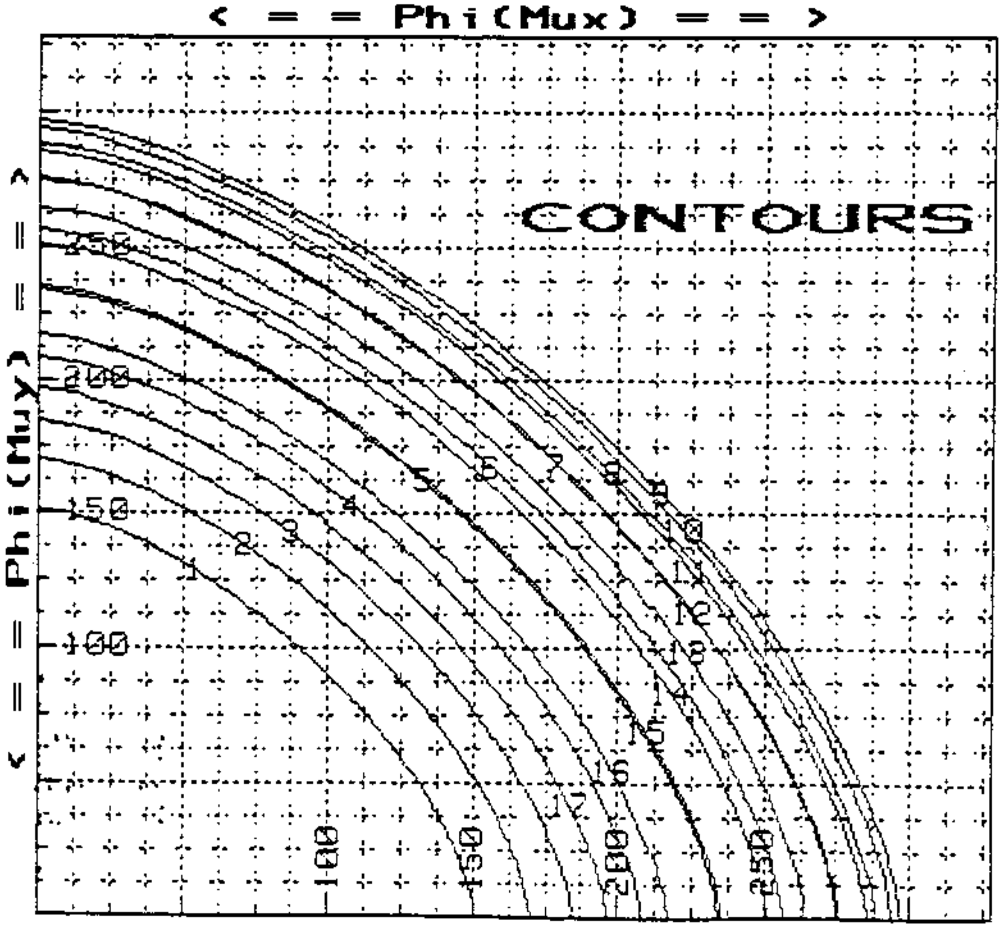

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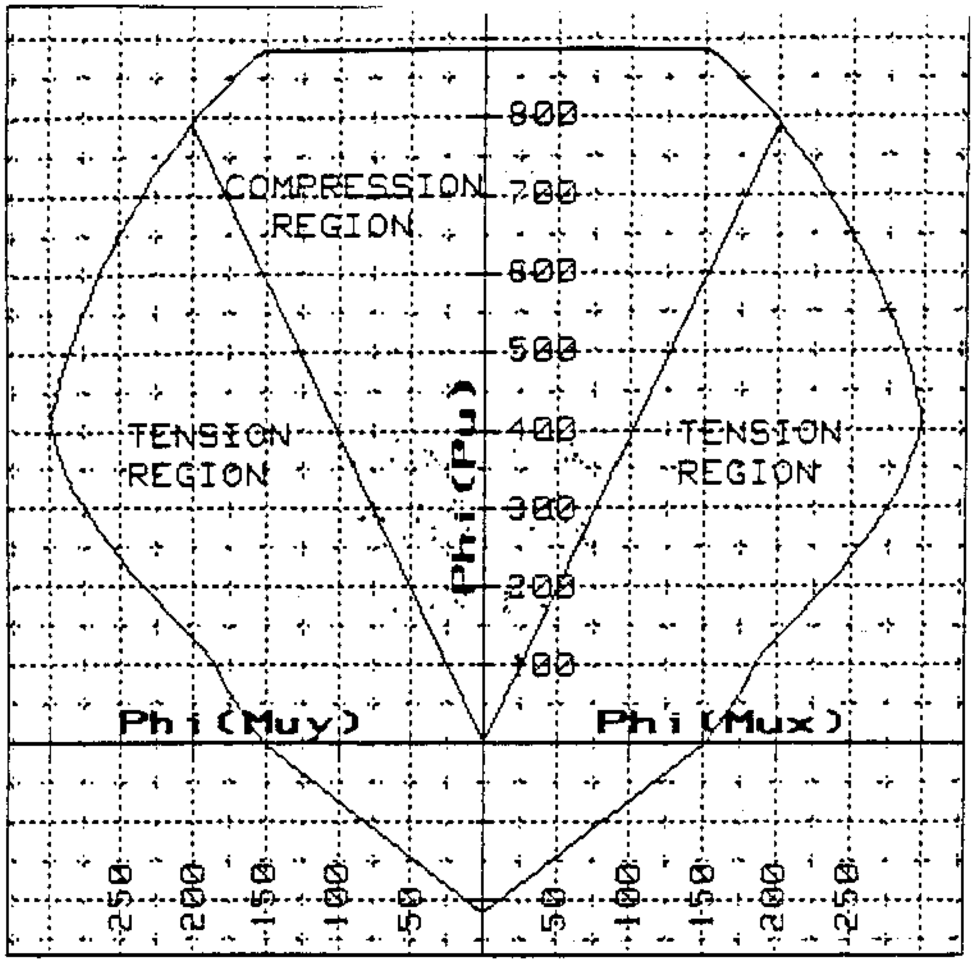

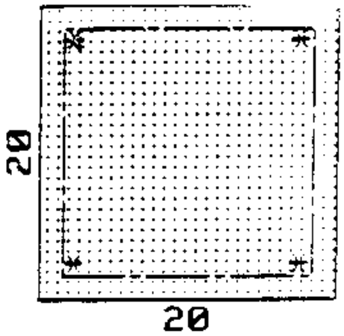

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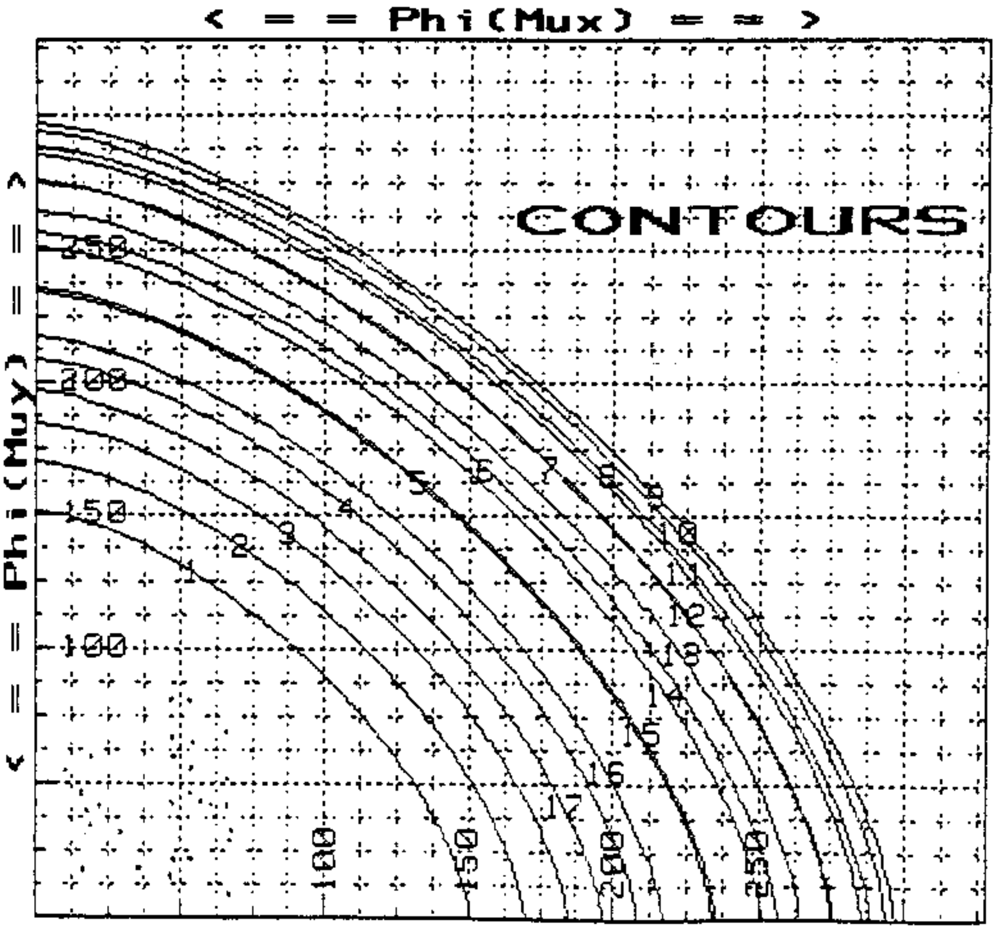

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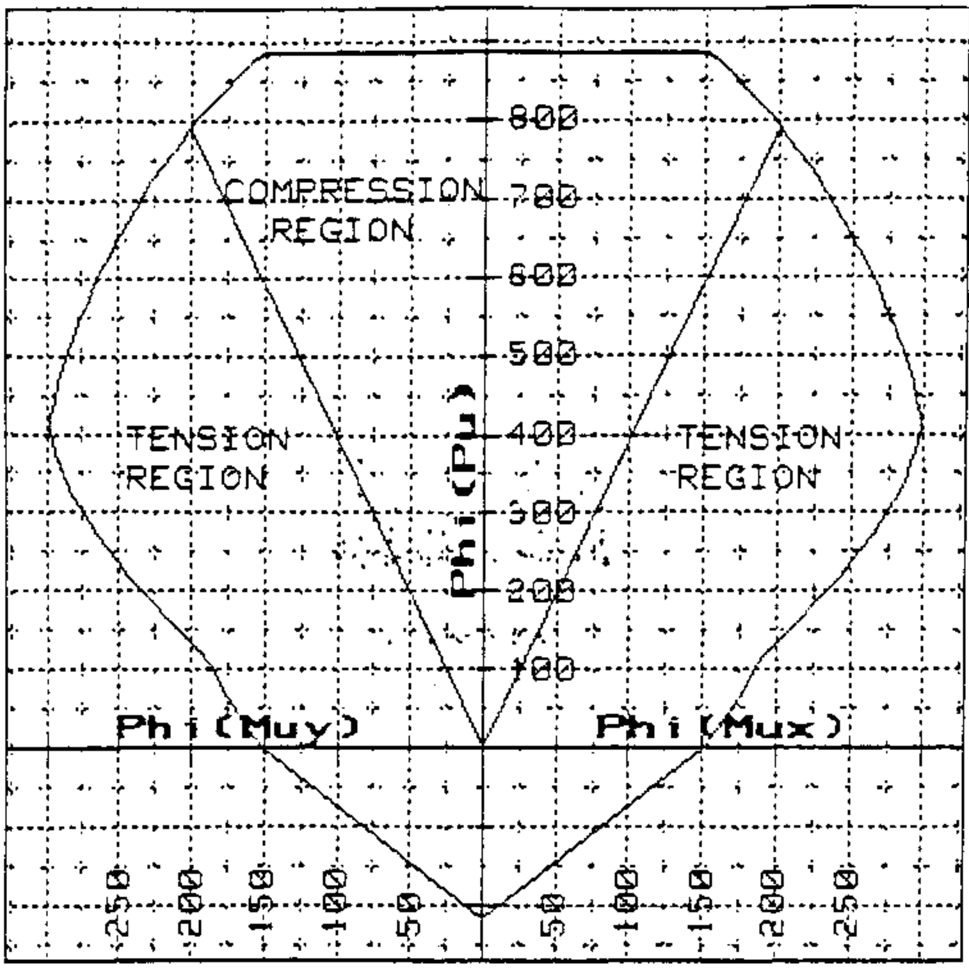

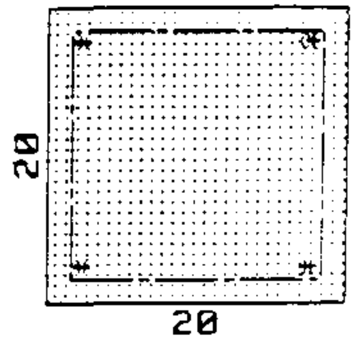

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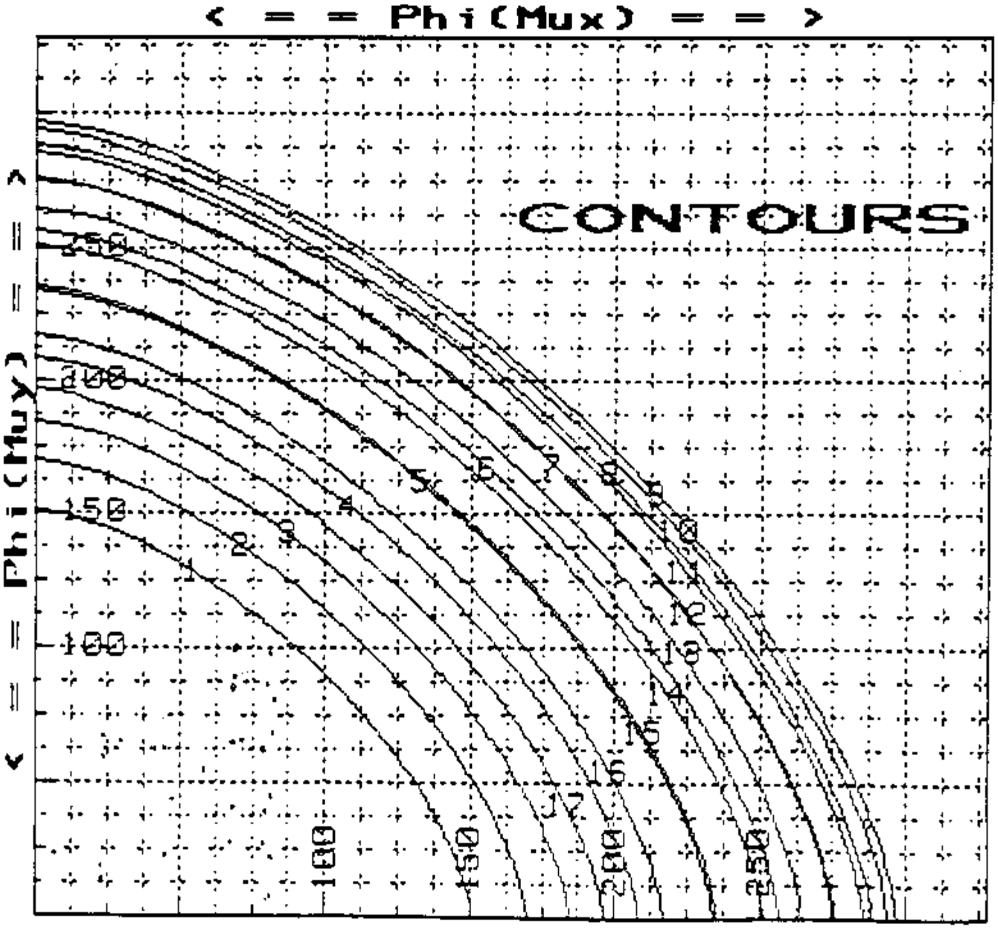

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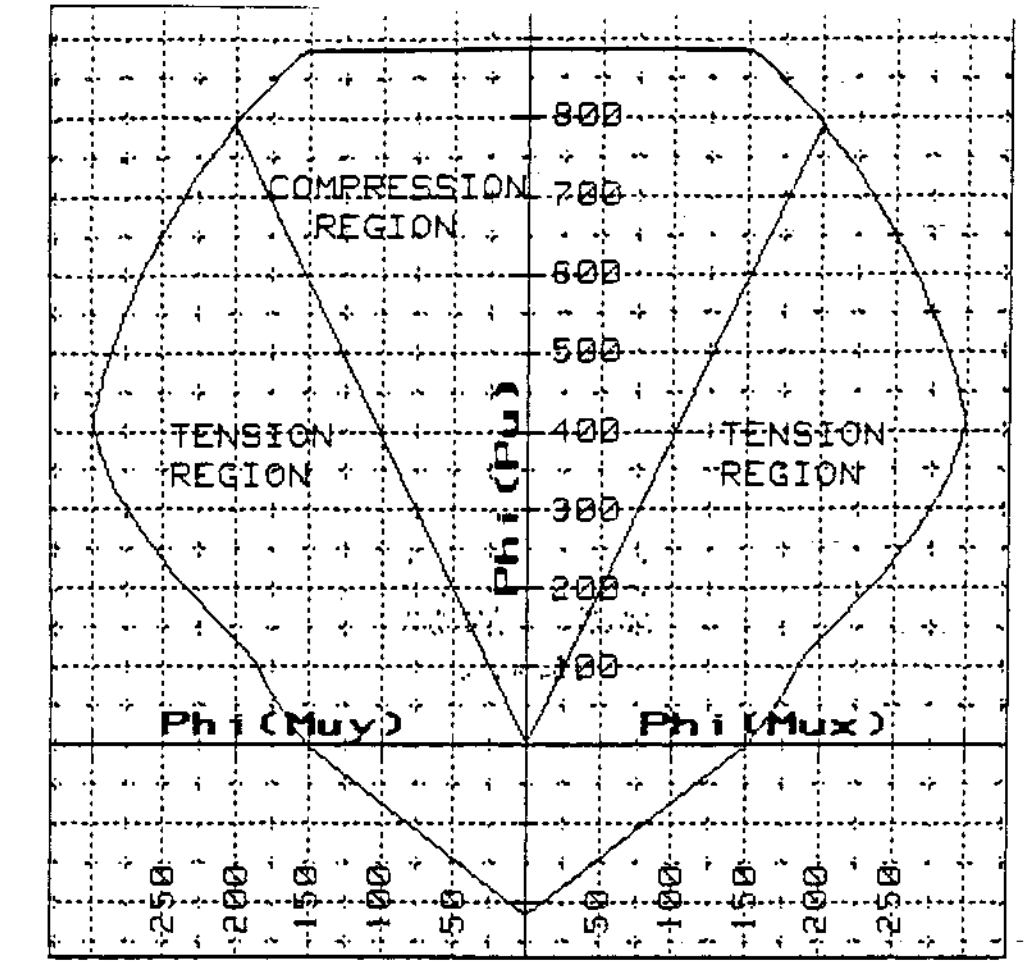

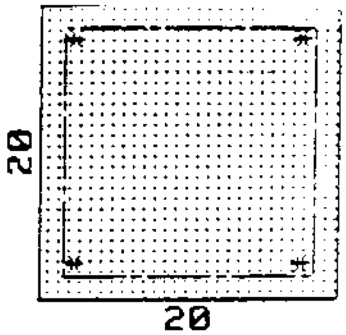

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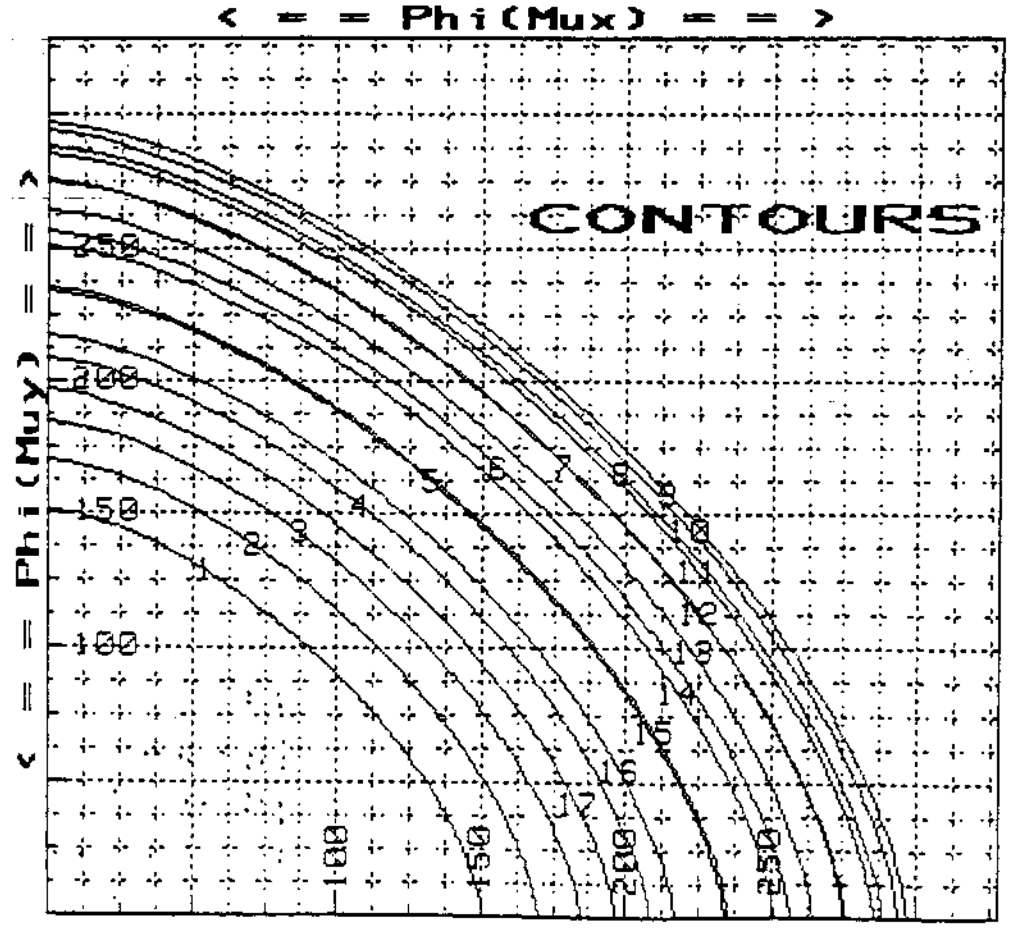

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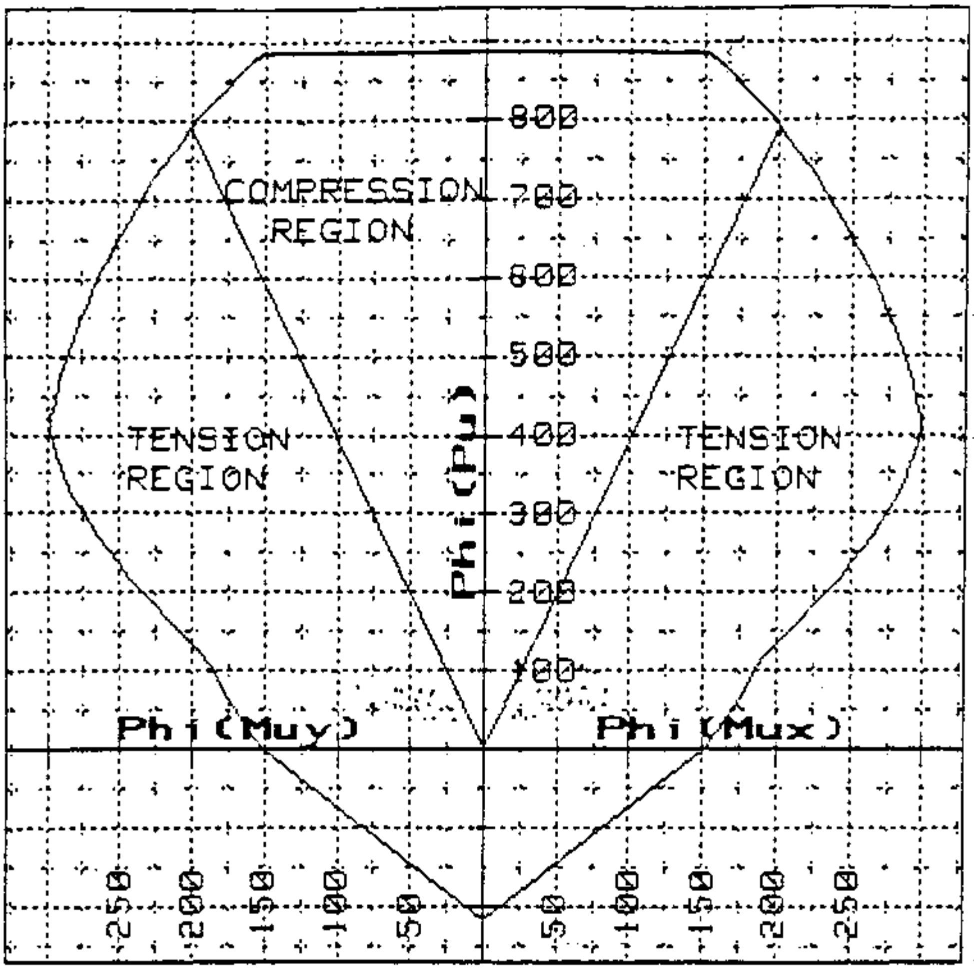

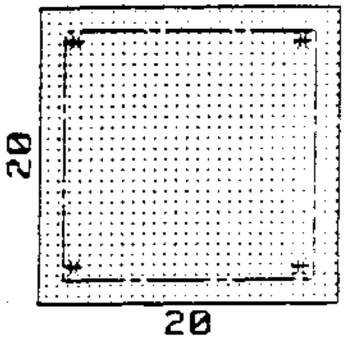

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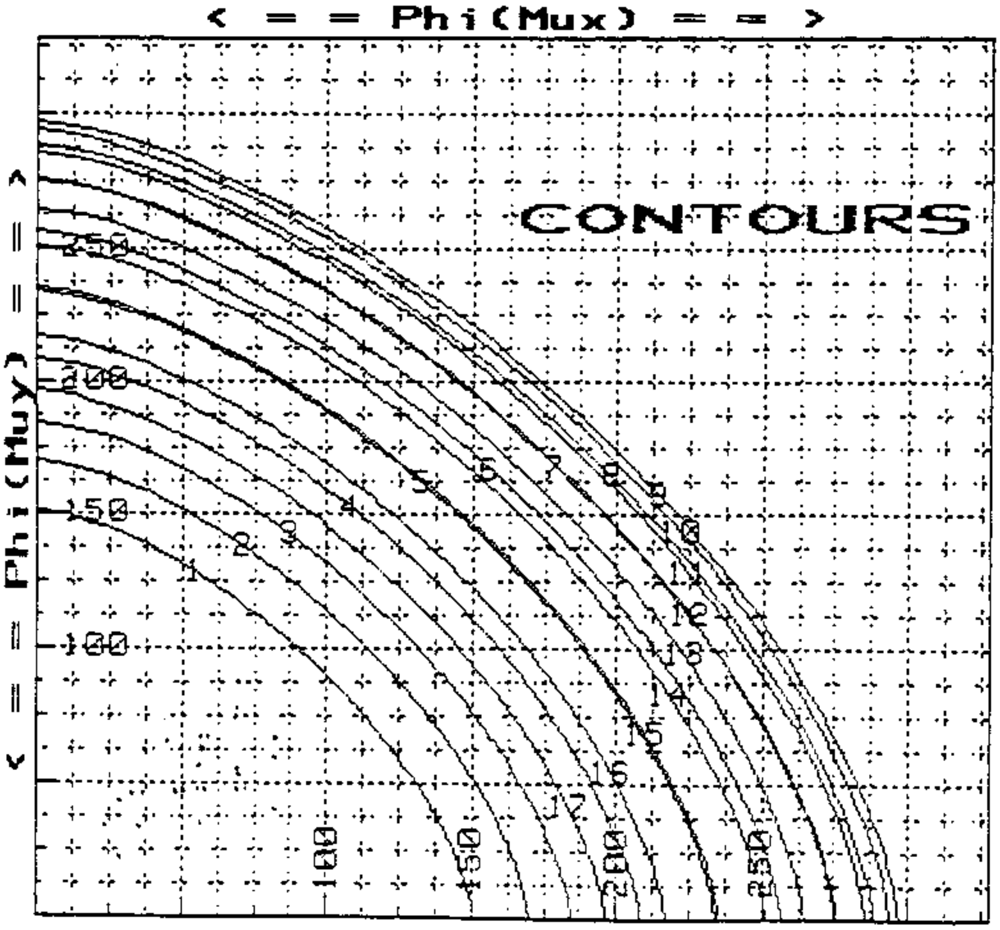

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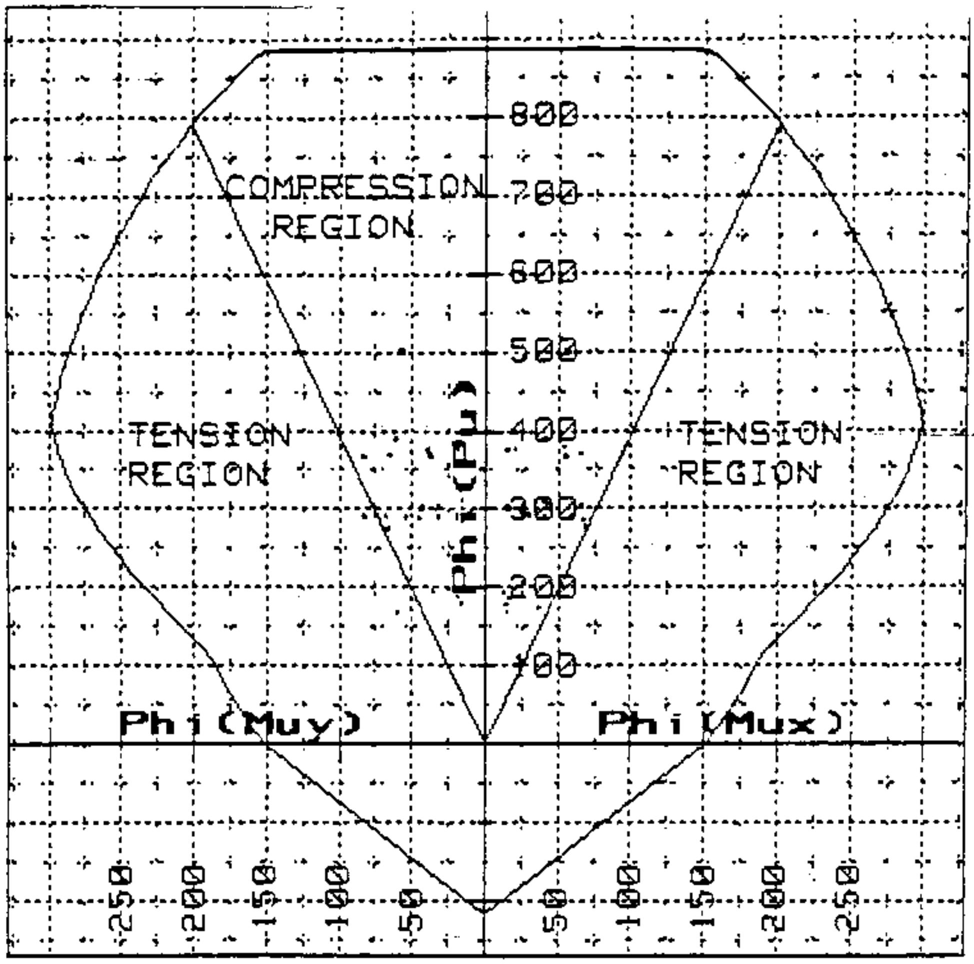

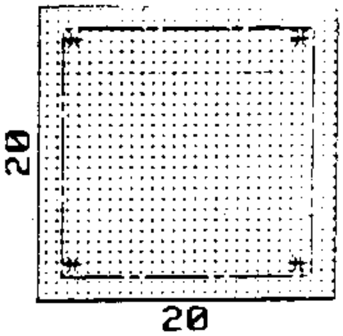

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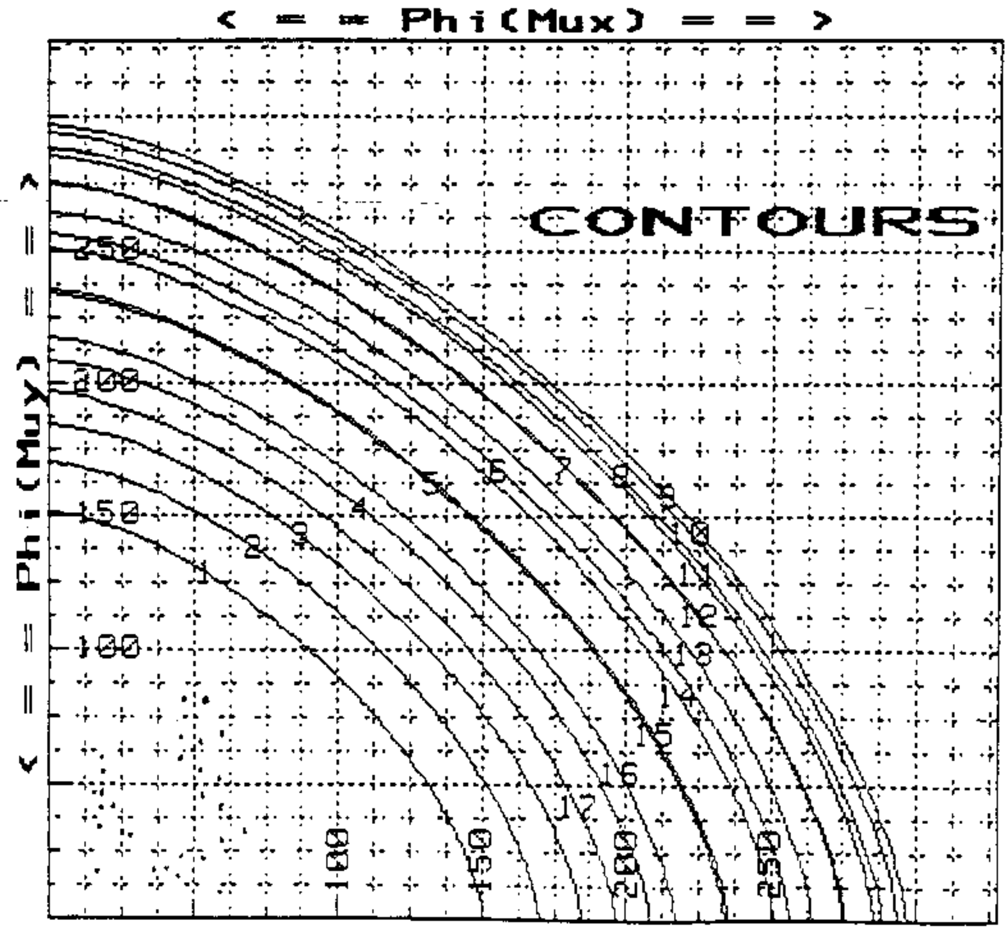

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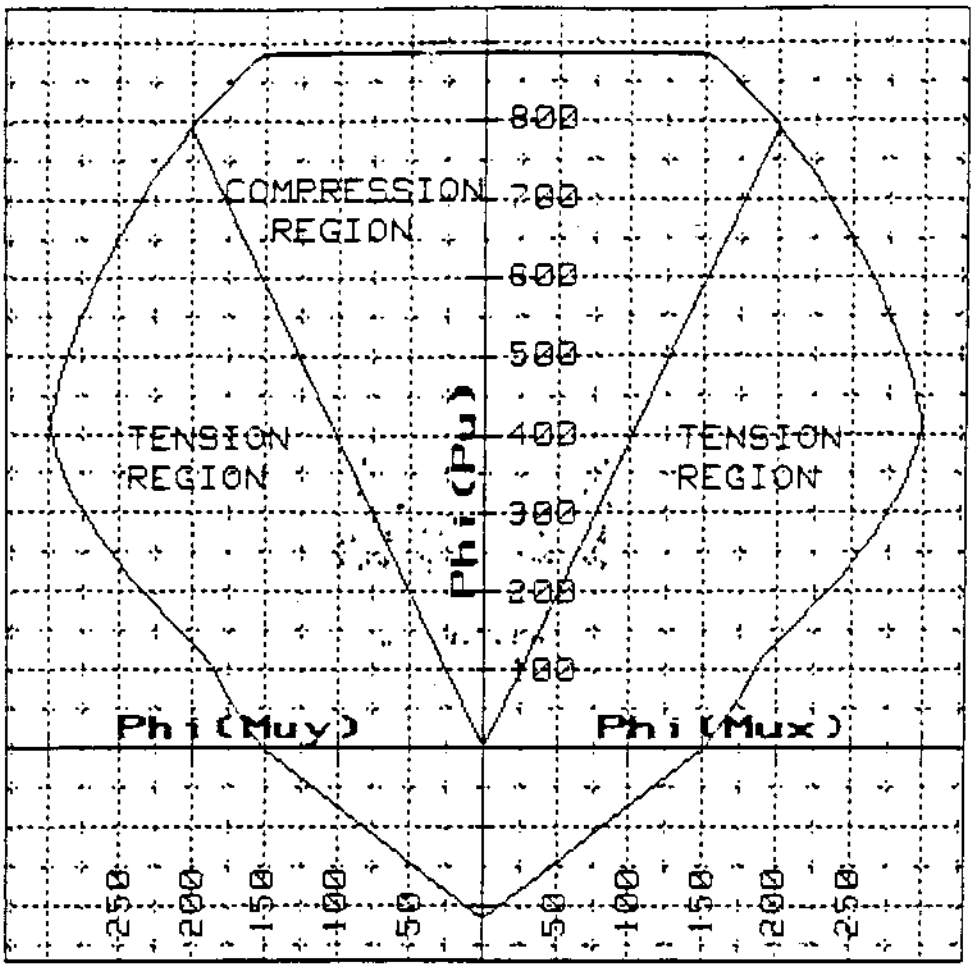

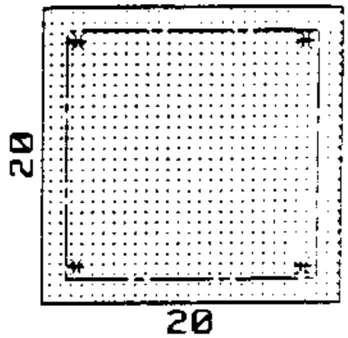

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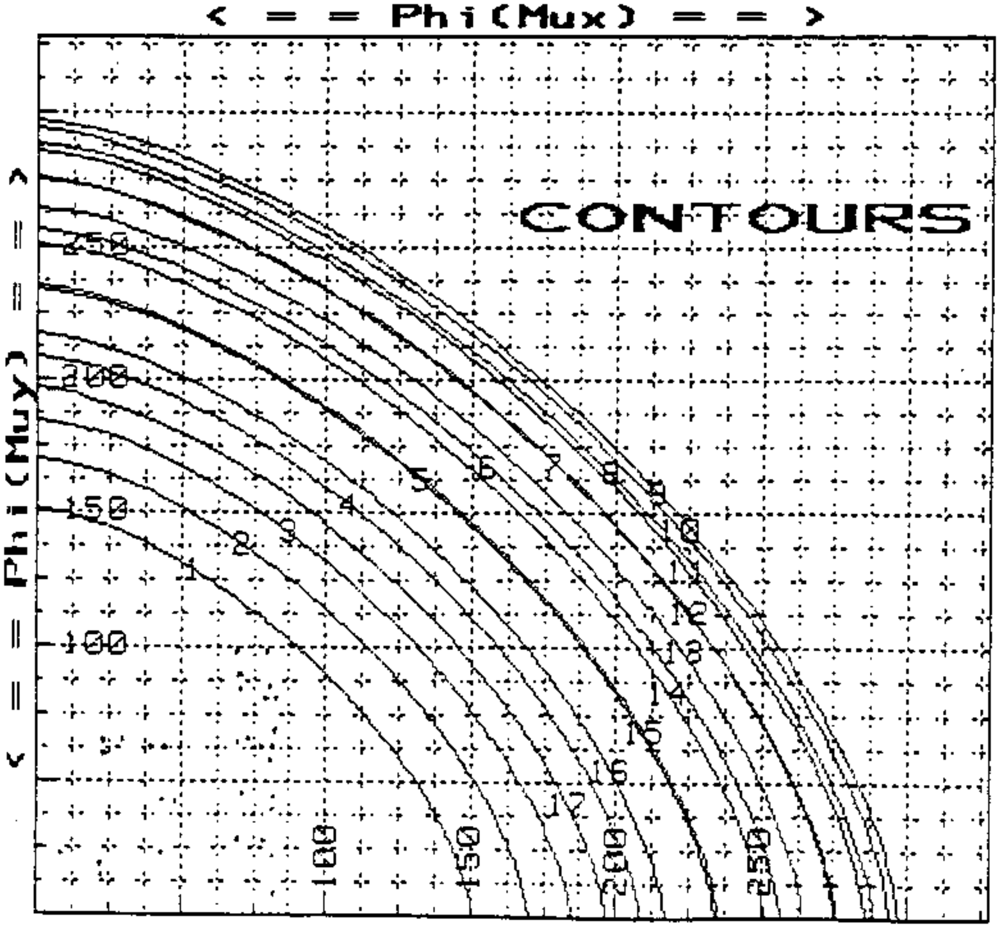

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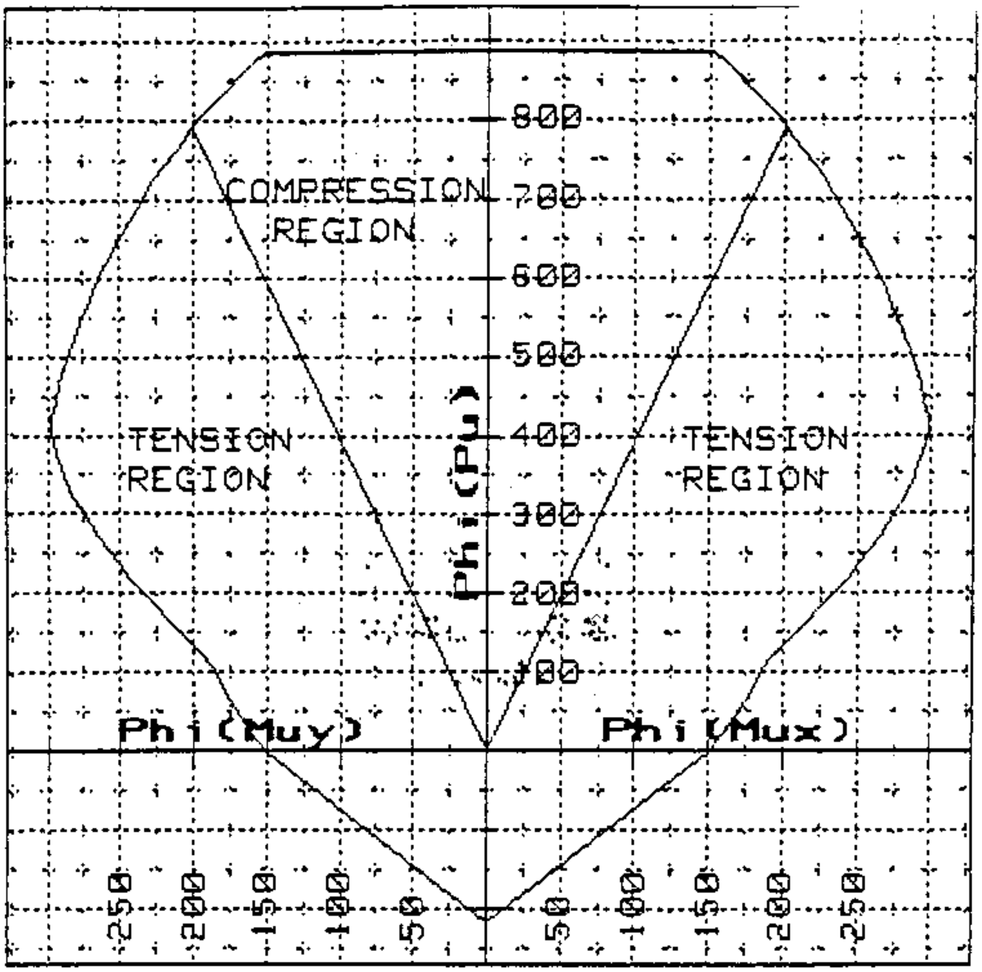

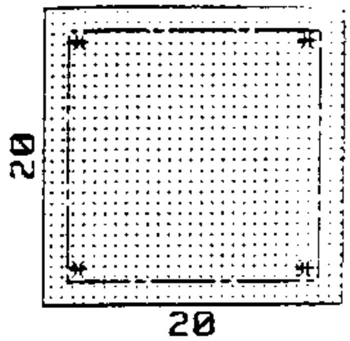

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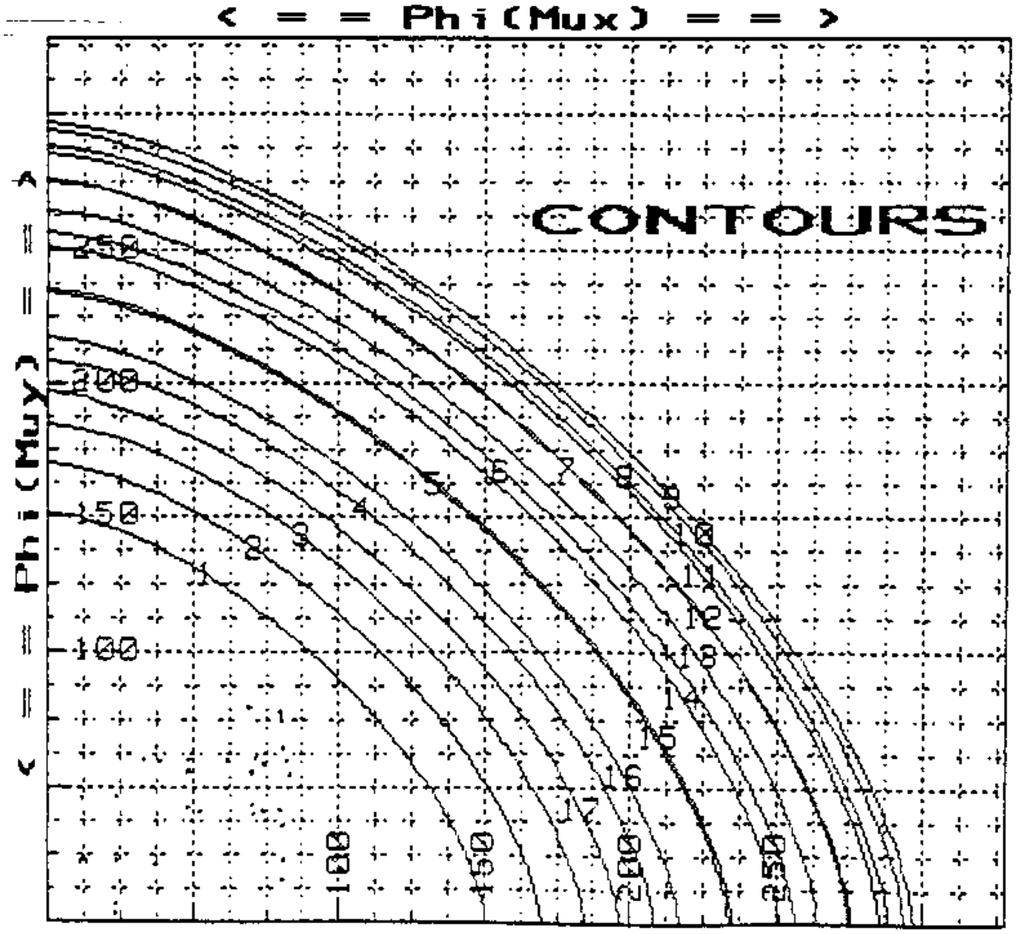

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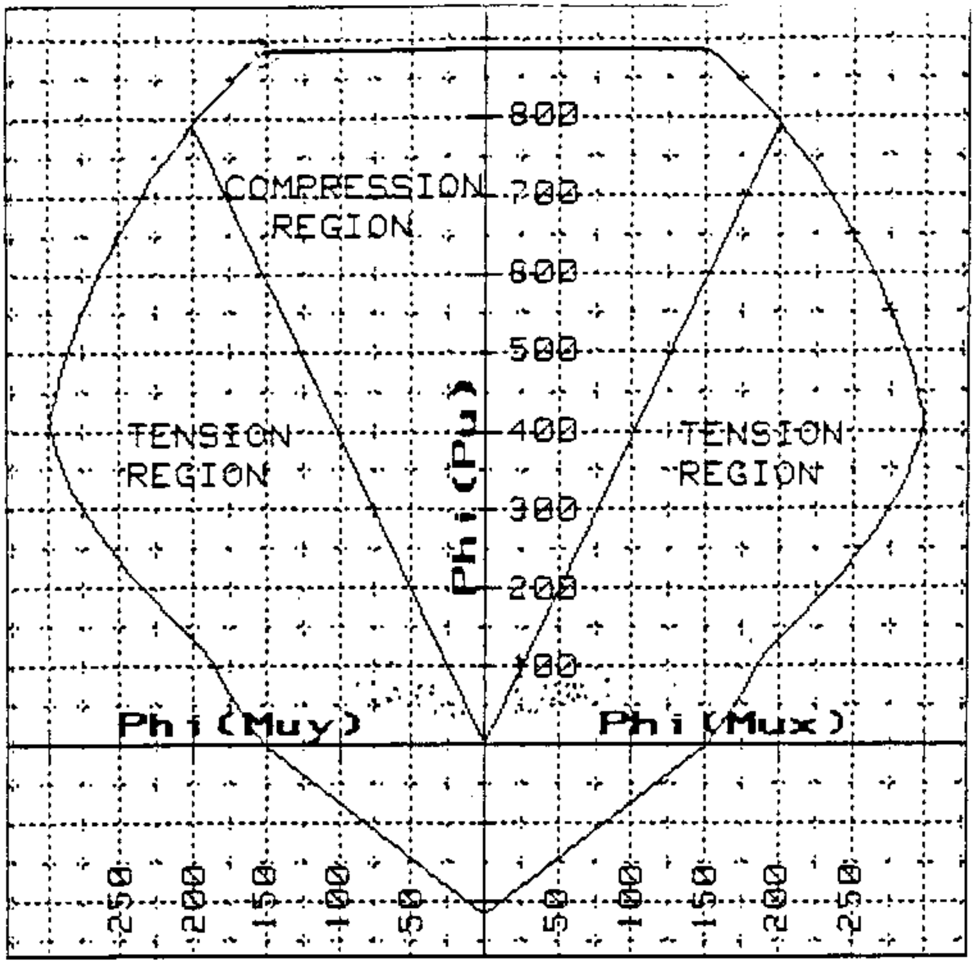
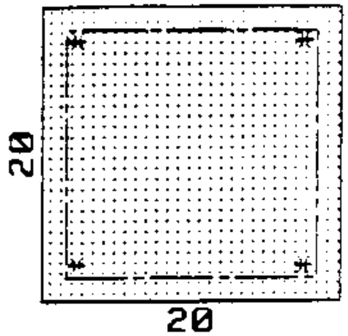
BT-3, CT-3





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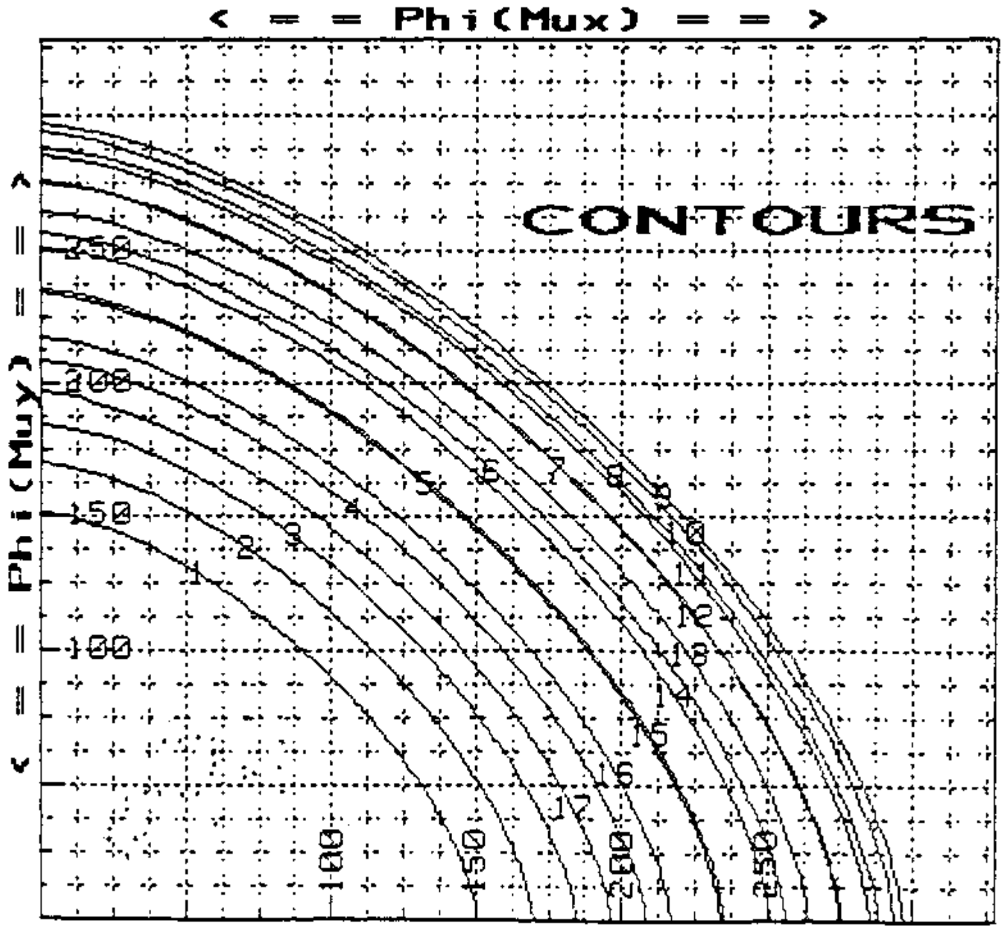

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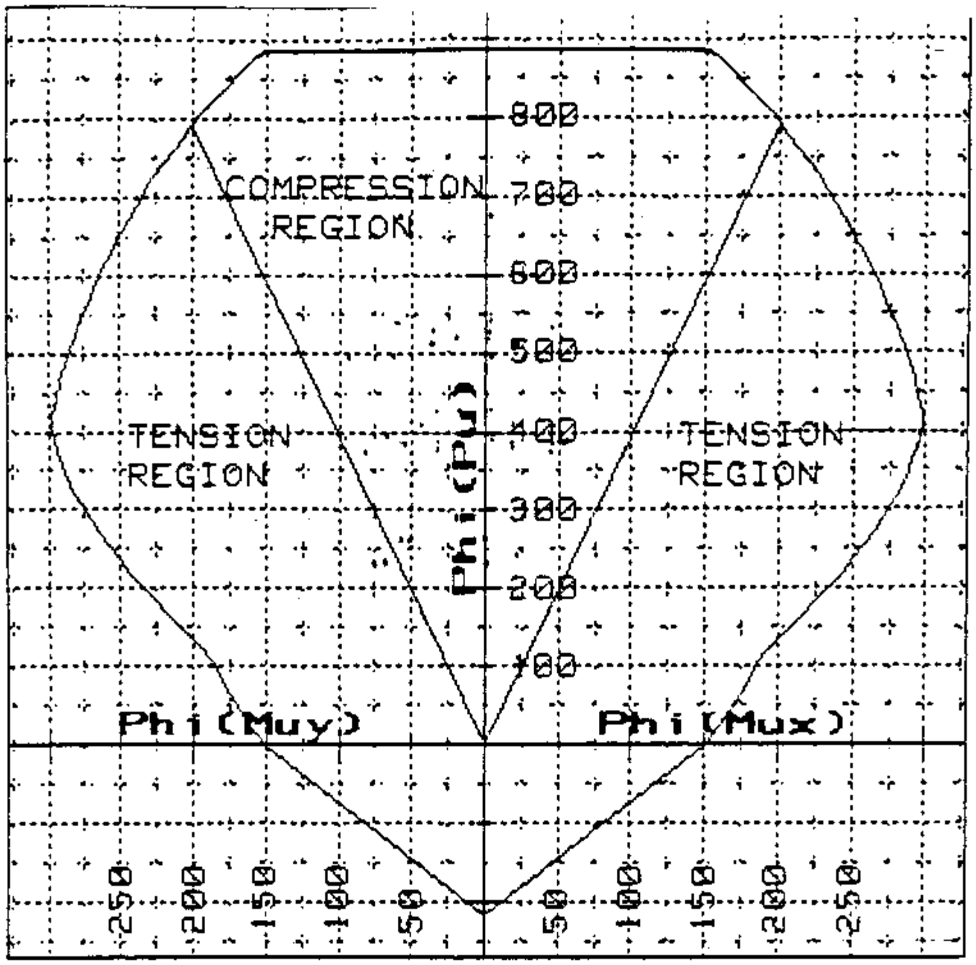

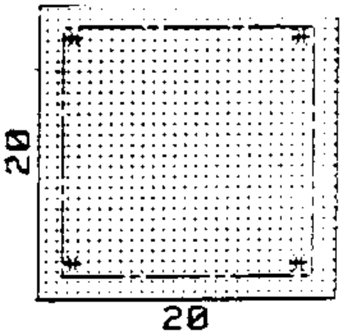

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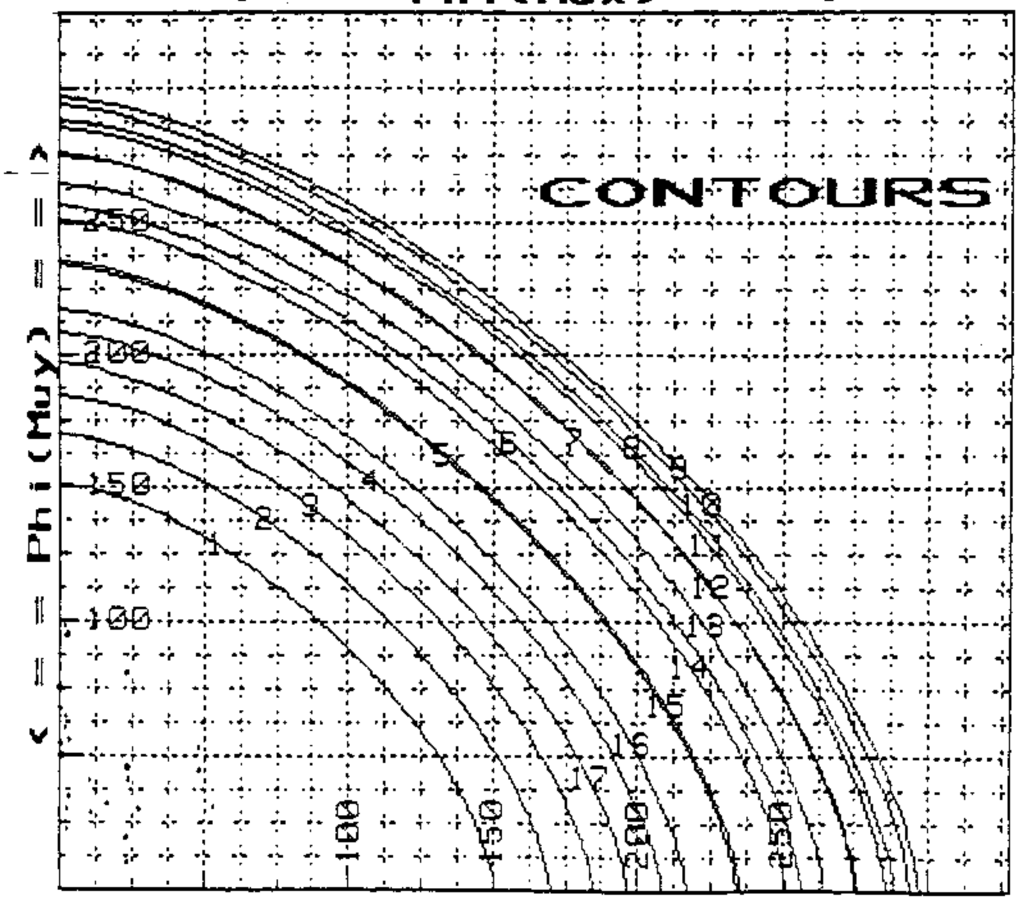

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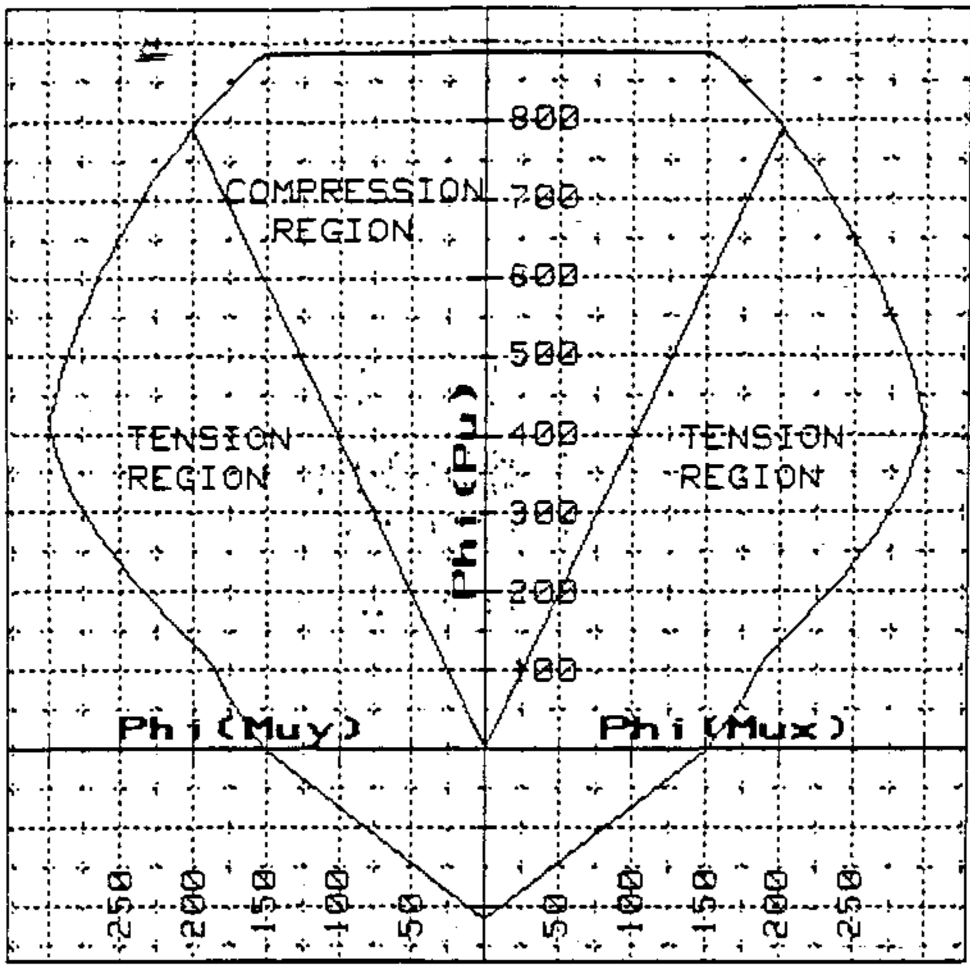
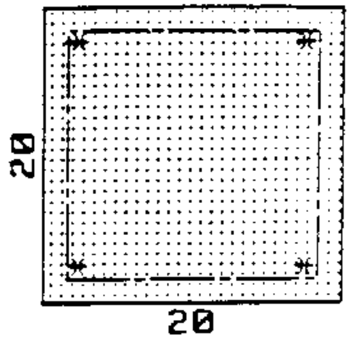


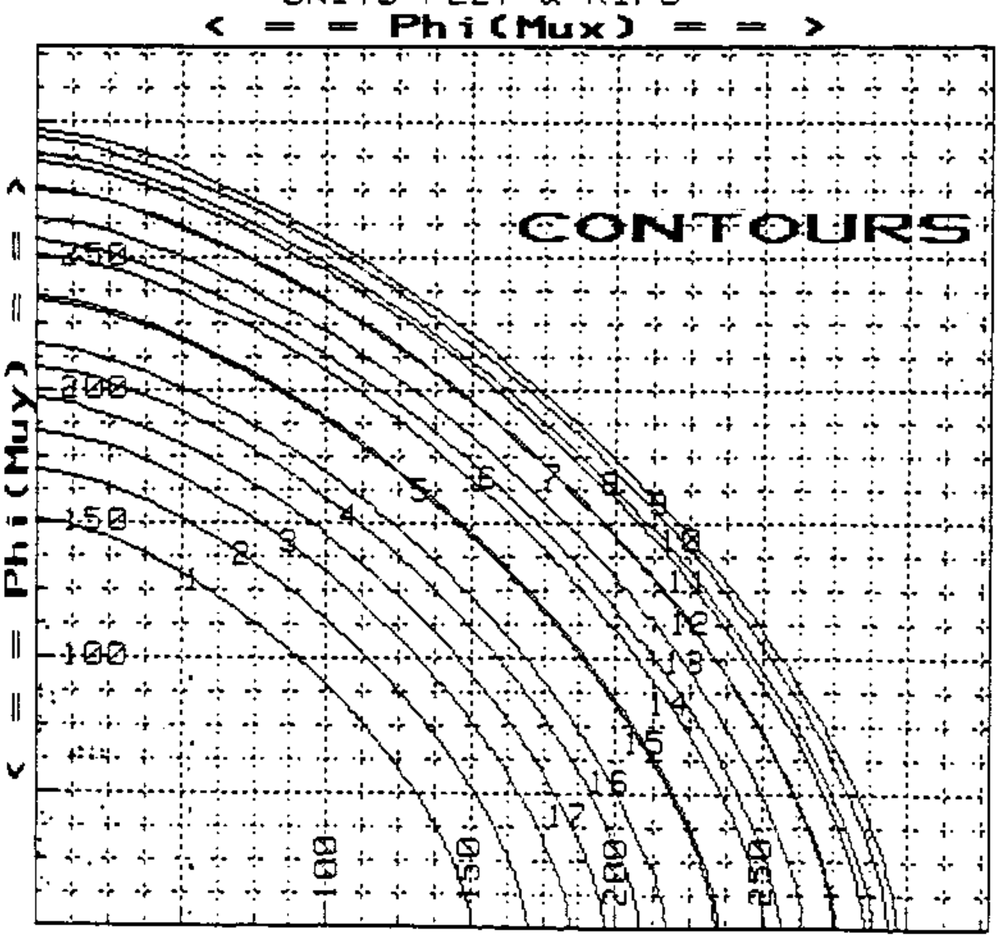


f'c= 4 KSI
~~Fy = 60 KSI~~
3 TIES
@ 18 O.C.
CORNER BARS
4 # 9
TOT.BARS= 4
Ast = 4
Rho = .01
KIPS & Ft.KIPS

PRINCIPAL AXES DIAGRAM

USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS



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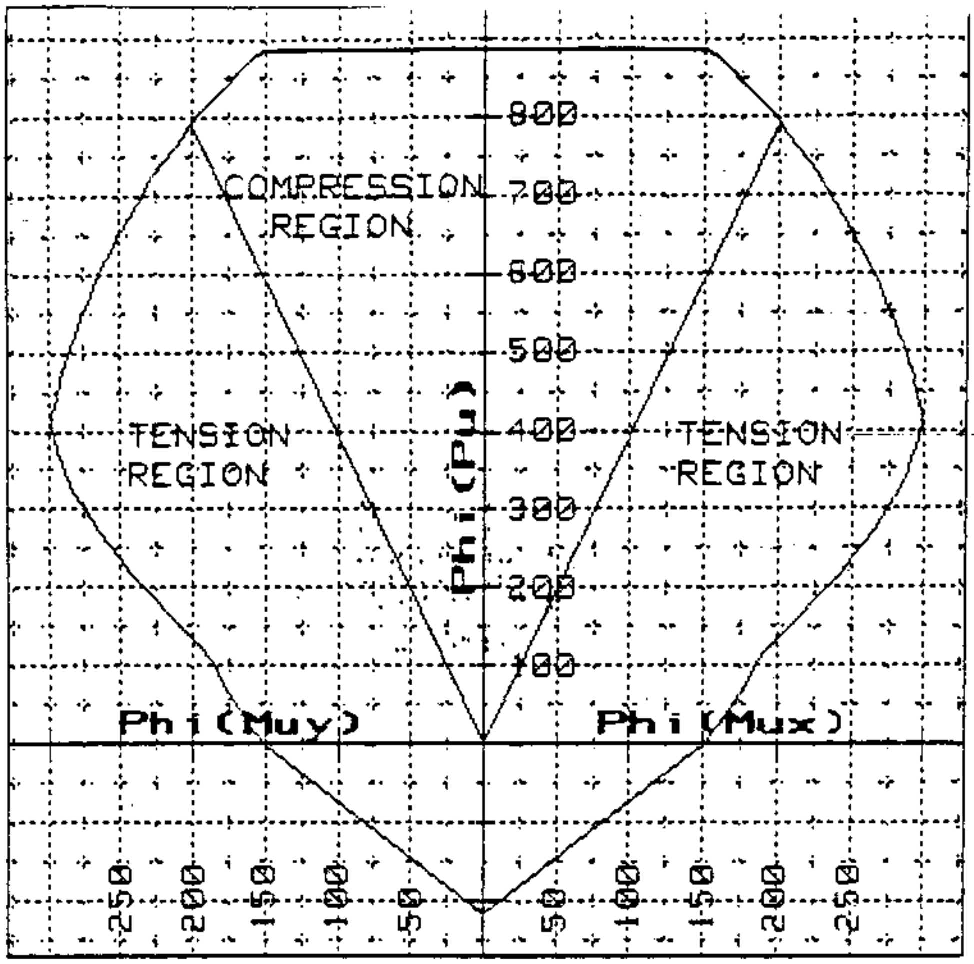

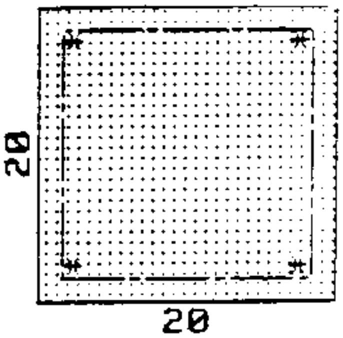

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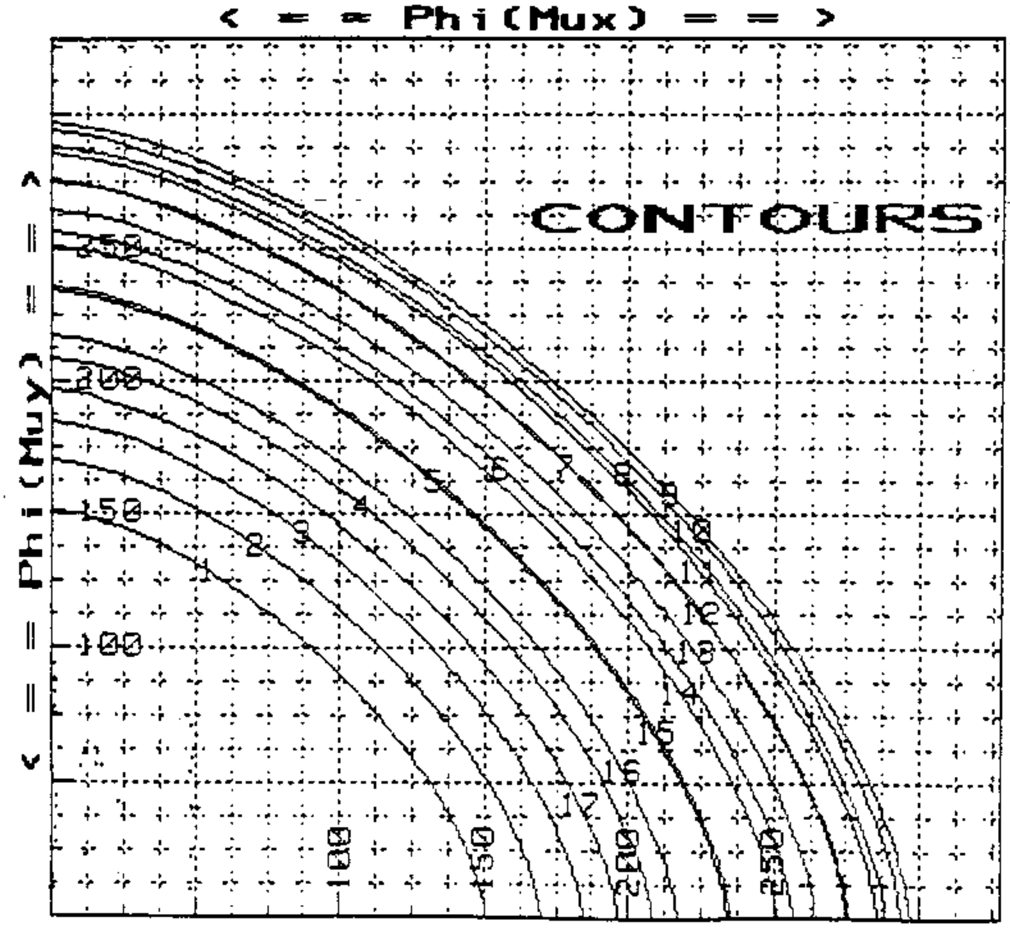

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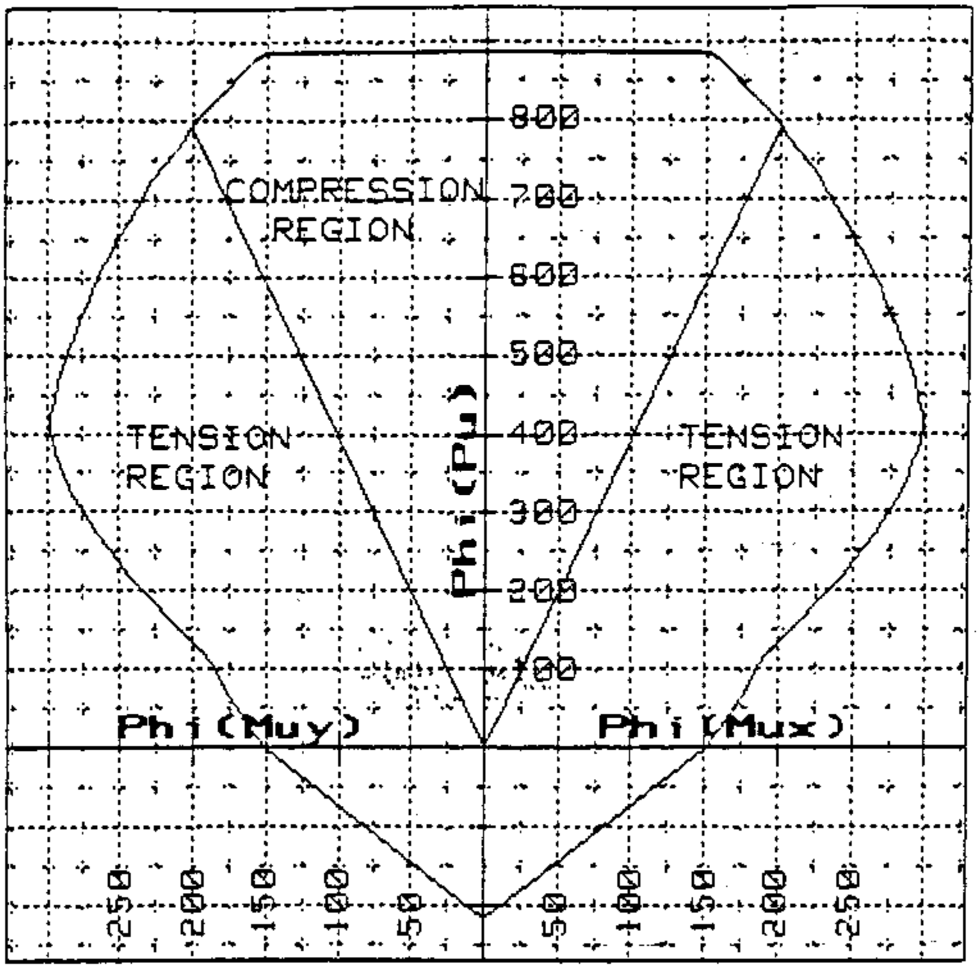
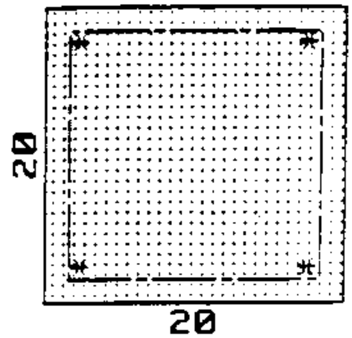
B5-3, C6-5





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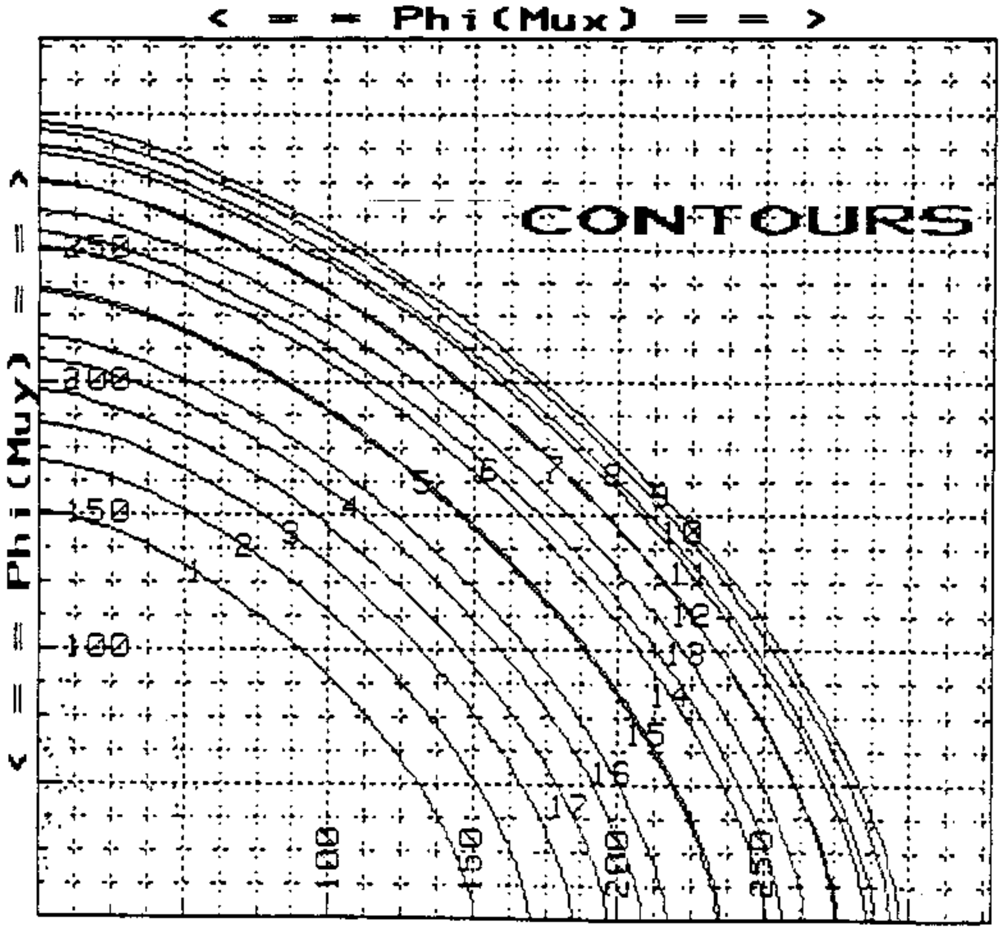

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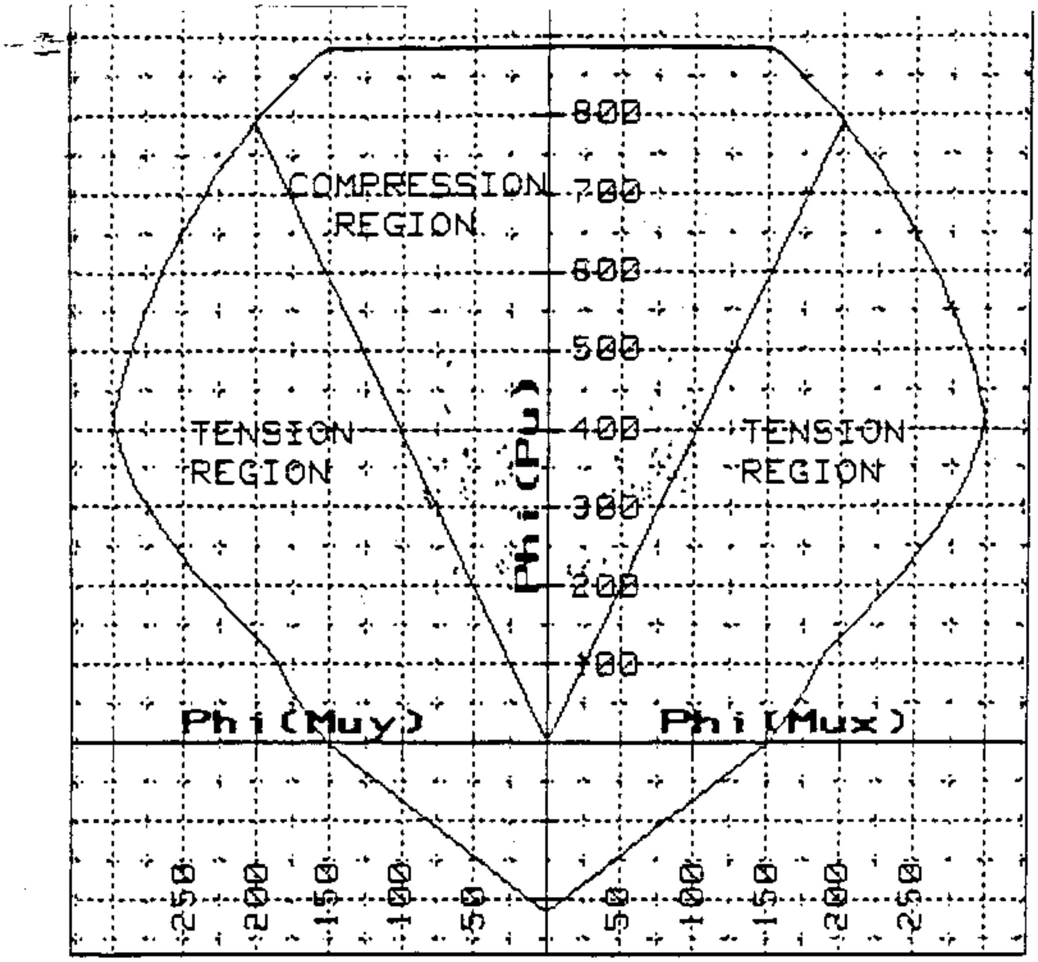
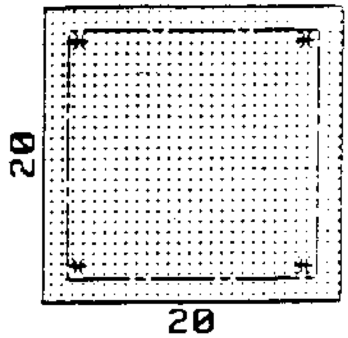
B8-4, C8-4





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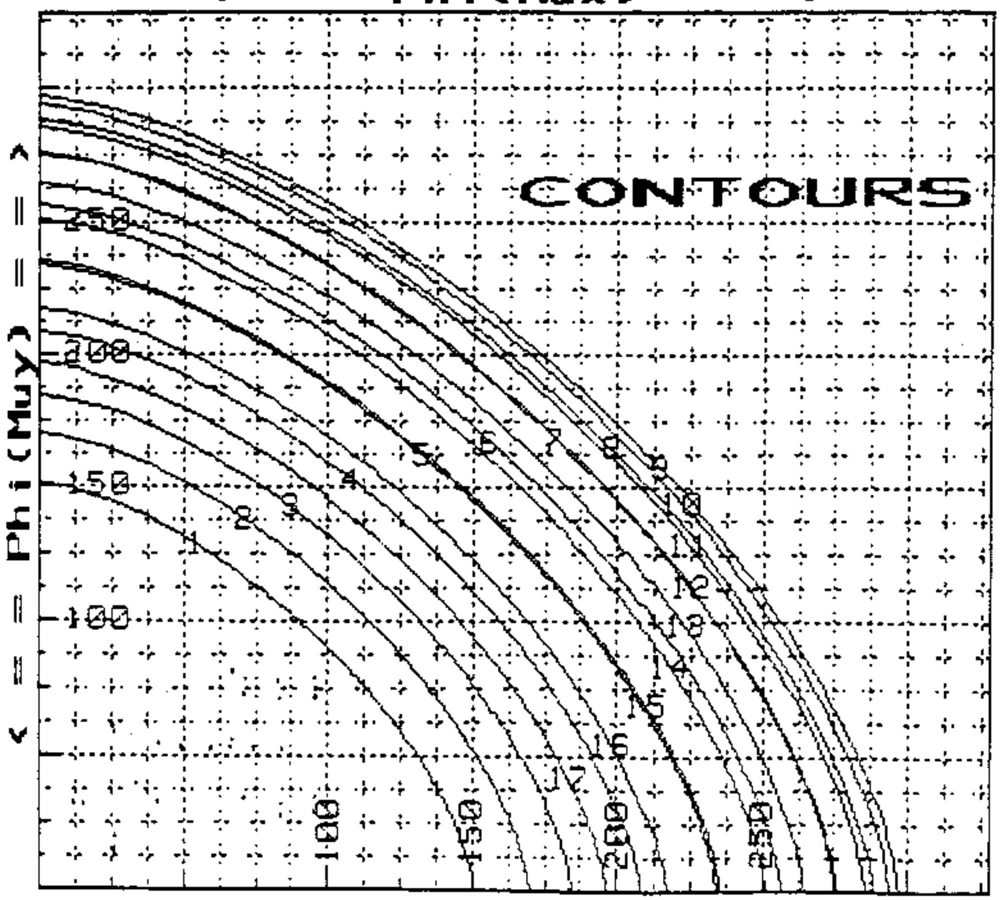
USE CLEAR COVER = 1.5 inches

UNITS FEET & KIPS

< = = Phi(Mux) = = >



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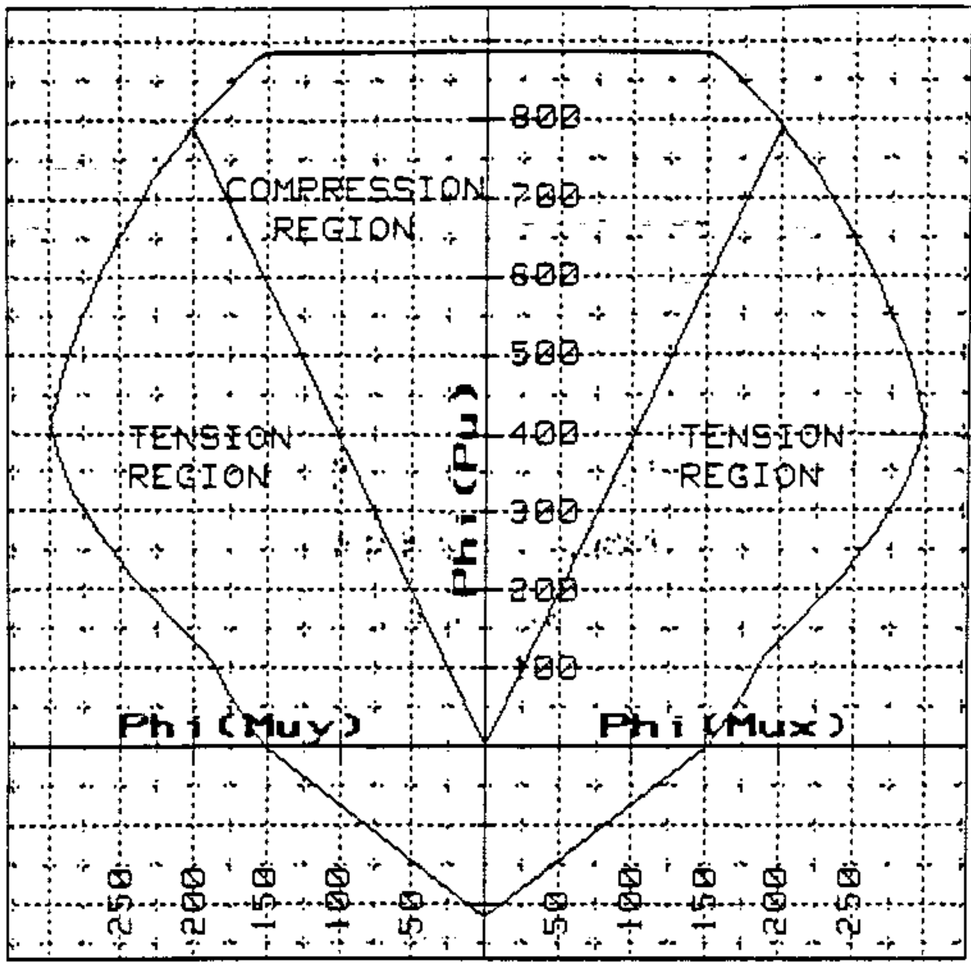

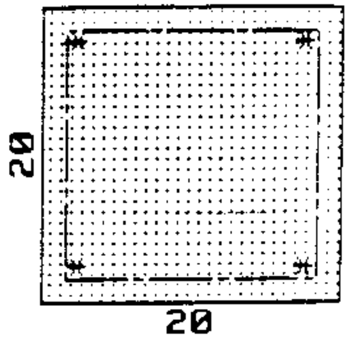

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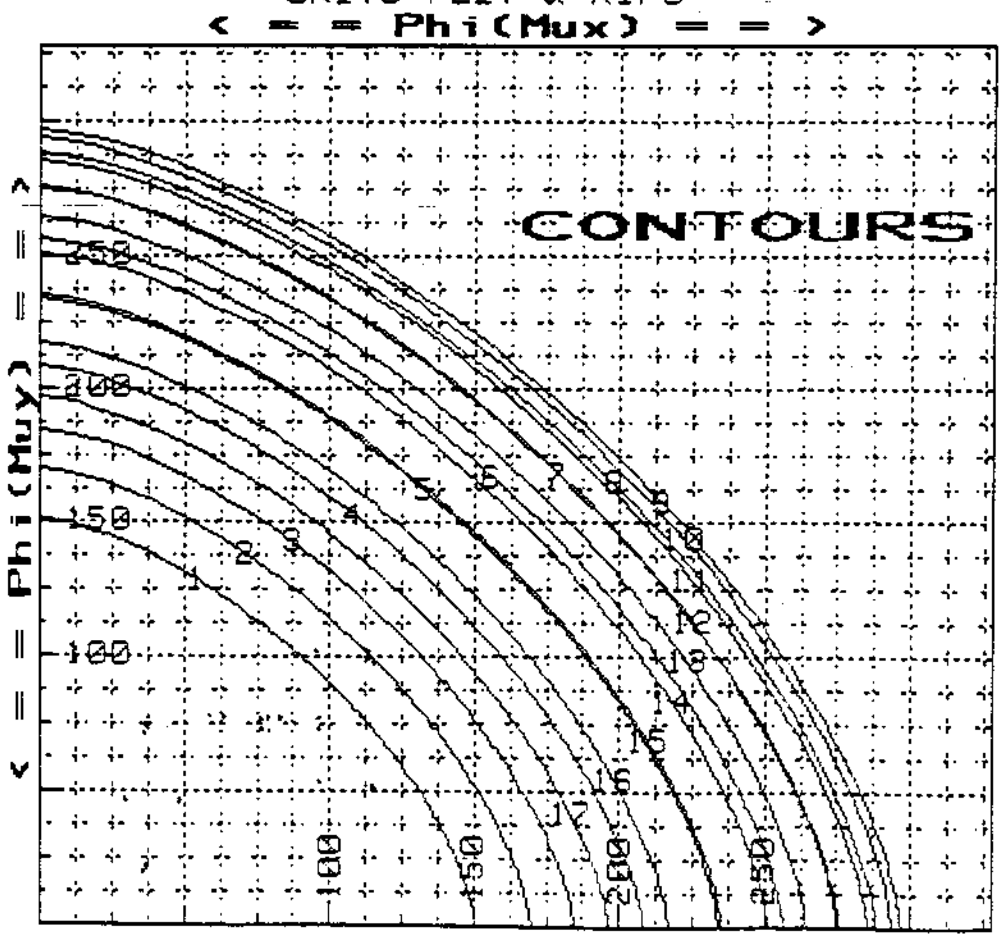

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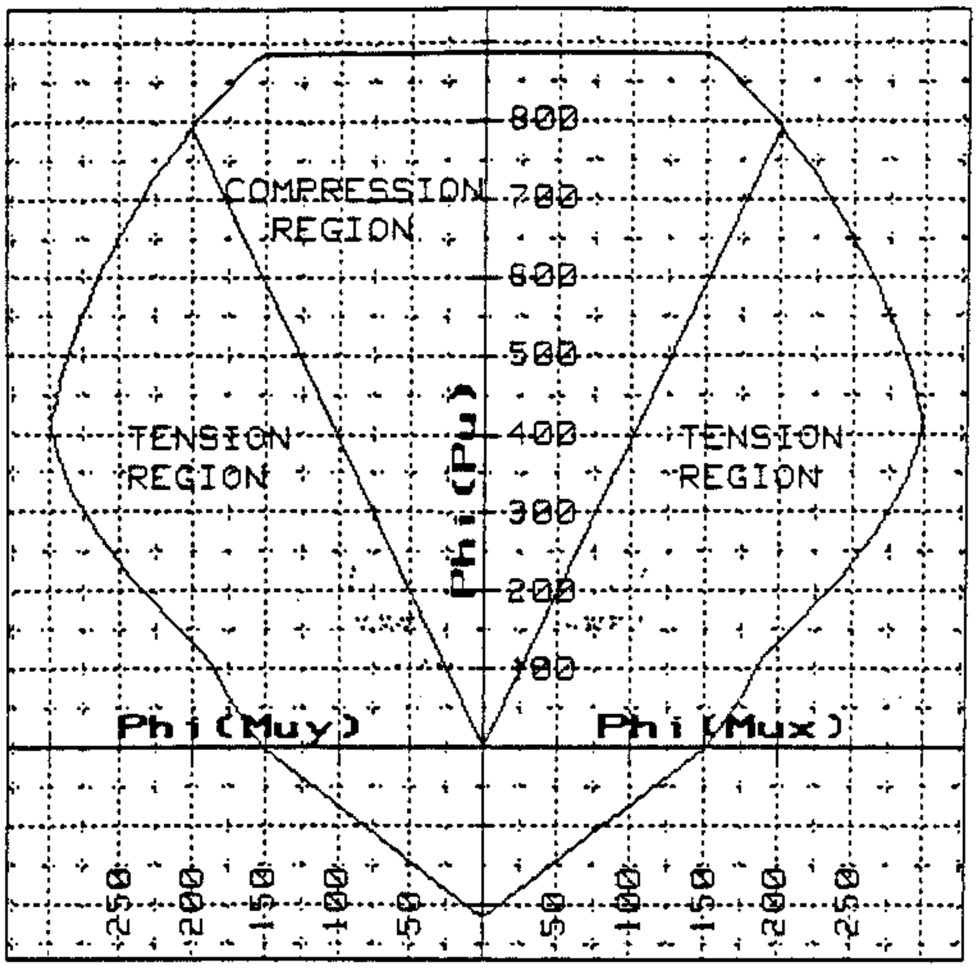
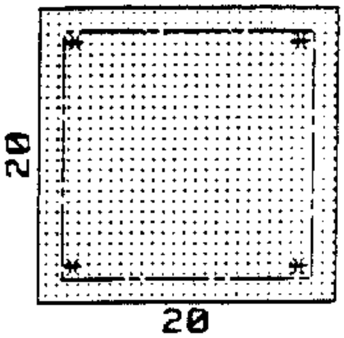
5A-2 C4-2 186





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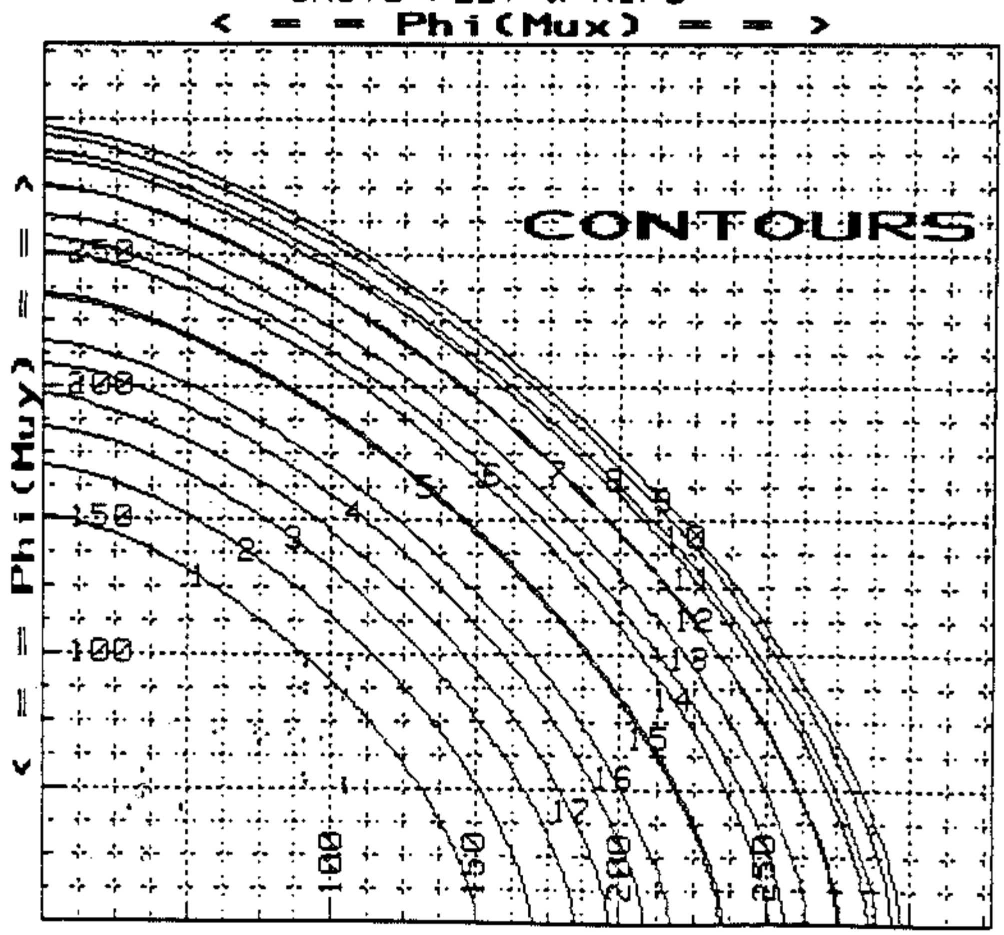

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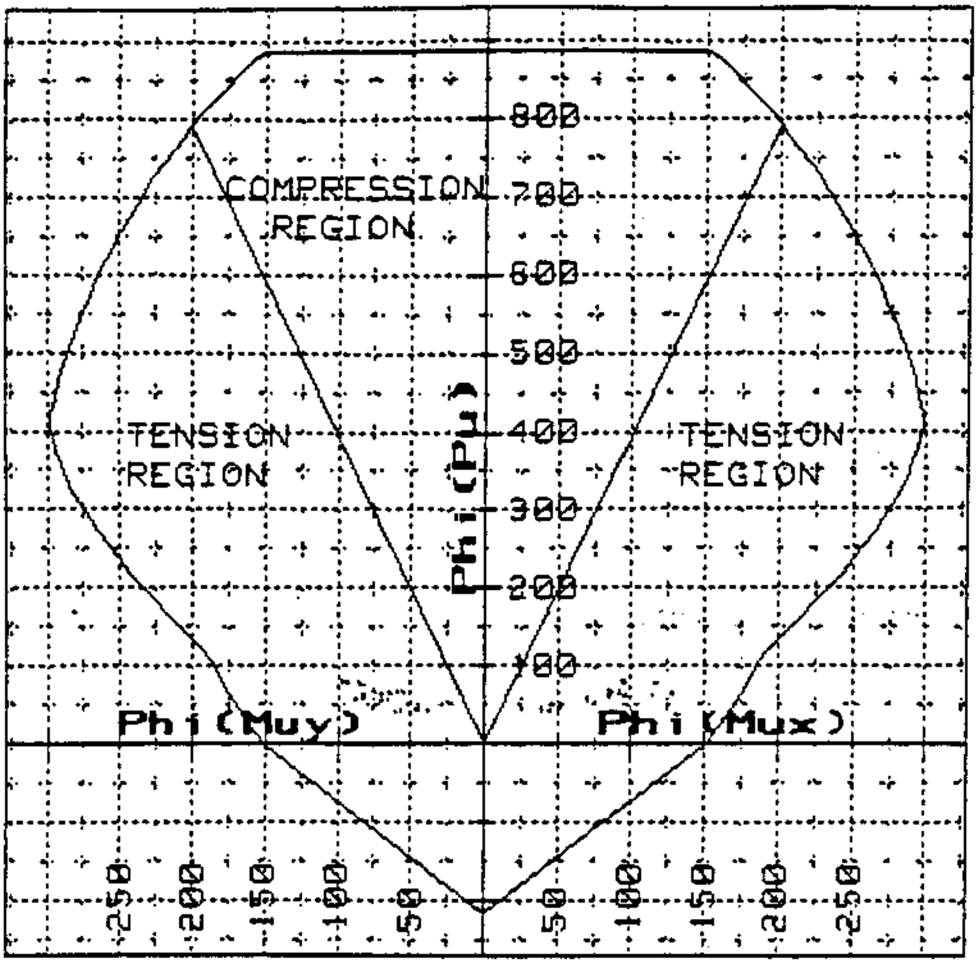
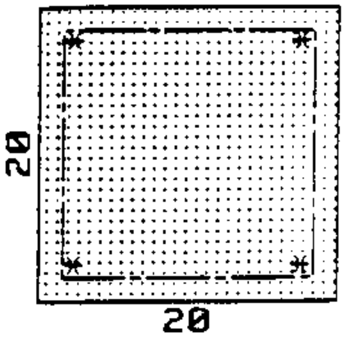
B-3, CA-3





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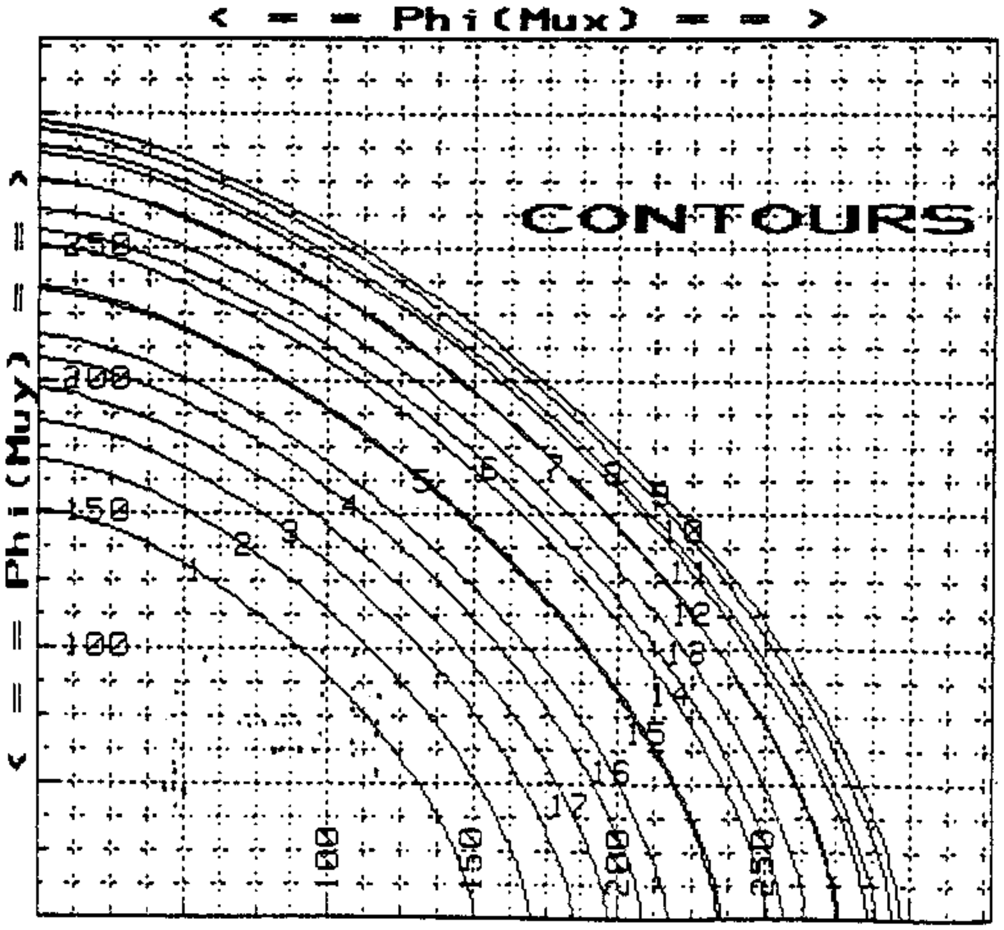

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FOUNDATION DESIGN

GRADE BEAMS

FRAMES A & D

LOADS

DEAD LOADS

G.B. SELF WT.	(2')(1')(0.150 KCF)	=	0.300 KLF
SLAB	(2.5')(4/12')(0.150 KCF)	=	0.125
PANEL	(4')(0.080 KSF)	=	0.320
GLAZING	(3.5')(0.005 KSF)	=	0.018
TOTAL			0.763 KLF = W_D

LIVE LOAD

SLAB	(2.5')(0.080 KSF)	=	0.200 KLF = W_L

$W_{TOTAL} = W_D + W_L = 0.763\text{ KLF} + 0.200\text{ KLF} = 0.963\text{ KLF}$

$W_u = 1.4W_D + 1.7W_L = (1.4)(0.763\text{ KLF}) + (1.7)(0.200\text{ KLF}) = 1.408\text{ KLF}$

FRAMES 1 & 9

LOADS

DEAD LOADS

G.B. SELF WT.	(2')(1')(0.150 KCF)	=	0.300 KLF
SLAB	(2.5')(4/12')(0.150 KCF)	=	0.125
PANEL	(7.5')(0.080 KSF)	=	0.600
TOTAL		=	1.025 KLF = W_D

LIVE LOAD

SLAB	(2.5')(0.080 KSF)	=	0.200 KLF = W_L

$W_{TOTAL} = W_D + W_L = 1.025\text{ KLF} + 0.200\text{ KLF} = 1.225\text{ KLF}$

$W_u = 1.4W_D + 1.7W_L = (1.4)(1.025\text{ KLF}) + (1.7)(0.200\text{ KLF}) = 1.775\text{ KLF}$

PIER DESIGN

ASSUMPTIONS

BEARING CAPACITY = 10 KSF AT 20'-0" BELOW FIN. FLR. (ELEV. 314'-0")
PIER SHAFT IS LATERALLY SUPPORTED BY THE SOIL
SHAFT DIAMETER MUST BE AT LEAST 1/3 OF THE BELL DIAMETER

DESIGN

ACI B.3.1-C PERMITS A SERVICE LOAD BEARING STRESS OF $0.3f'_c$
$\therefore f_c = 0.3(4000\ PSI) = 1200\ PSI = 1.200\ KSI$

SHAFT DIA.	SHAFT CAPACITY (1.2 KSI)
1'-6"	305.363^K
2'-0"	542.867^K
2'-6"	848.230^K
3'-0"	1221.451^K

BELL DIA.	BELL BEARING CAPACITY (10 KSF)
6'-0"	282.743^K
6'-6"	331.831^K
7'-0"	384.845^K
7'-6"	441.786^K
8'-0"	502.655^K

NAME	P_{MAIN} (KIPS)	P_{PERP} (KIPS)	$P_{SUBTOTAL}$	G.B'S	P_{TOTAL}	SHAFT DIA.	BELL DIA.	V_{MAIN} (K-FT)	$M_{PERP.}$ (K-FT)	COMP. STRESS (KSI)
A1,D1	136.977	141.101	278.078	33.709	311.365	3'-0"	6'-6"	27.857	42.229	0.490
A2,D2	272.992	119.568	392.560	23.112	415.672		7'-6"	0.187	28.835	0.484
A3,D3	170.100	119.642	289.742	17.334	307.076		6'-6"	17.974	28.883	0.424
A4,D4	170.330	119.642	289.972	17.334	307.306		6'-6"	19.501	28.883	0.429
A5,D5	270.962	119.642	390.604	23.112	413.716		7'-6"	0.0	28.883	0.482
[illegible]	170.330	119.642	289.972	17.334	307.306		6'-6"	19.501	28.883	0.429
[illegible]	170.100	119.642	289.742	17.334	307.306		6'-6"	17.974	28.883	0.425
A8,D8	272.992	119.642	392.634	23.112	415.746		7'-6"	0.187	28.883	0.485
[illegible]	136.977	138.553	275.530	33.790	309.320		6'-6"	27.857	35.246	0.469
[illegible]	115.616	208.931	324.547	26.950	351.497		7'-0"	21.915	12.425	0.435
[illegible]	257.864	180.214	438.078	0.0	438.078		8'-0"	4.305	18.056	0.490
[illegible]	173.431	156.521	329.952	0.0	329.952		6'-6"	17.343	19.605	0.421
B4,C4	172.802	156.521	329.323	0.0	329.323		6'-6"	18.323	19.605	0.423
B5,C5	283.806	156.521	440.327	0.0	440.327		8'-0"	0.072	19.605	0.484
[illegible]	172.848	156.521	329.369	0.0	329.369		6'-6"	18.313	19.605	0.423
[illegible]	172.621	156.521	329.142	0.0	329.369		6'-6"	16.794	19.605	0.419
[illegible]	287.140	156.521	443.661	0.0	443.661		8'-0"	0.924	19.605	0.490
[illegible]	141.443	189.244	330.687	26.950	357.637	3'-0"	7'-0"	27.180	24.665	0.487

CONCLUSION

This project has provided needed practical experience in the design of a reinforced concrete building. From the initial design stages to the detailing of the connections, new knowledge was obtained and applied throughout the project. I am sure that this experience as well as my previous experiences at Oklahoma State University will prove to be invaluable to me in the future. As the old saying goes, "there is no teacher like experience", how true.

Very special thanks to my wife, my parents, and to Professor Louis O. Bass.